VOLUME 87 NO. PO2
JULY 1961

JOURNAL of the

Power Division

PROCEEDINGS OF THE

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The index for 1959 was published as ASCE Publication 1960-10 (list price $2.00); indexes for previous years are also available.

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Vol. 87 No. PO 2

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

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STUDIES FOR THE LOWER MONUMENTAL POWER PLANT[a]

By Benjamin F. Smith,[1] M. ASCE

SYNOPSIS

The economic development of water resources for power, navigation, flood control, and other beneficial uses requires the application of both a high degree of technical knowledge and sound judgment in many fields of engineering. The planning and design of hydroelectric power plants provide an excellent example of professional accomplishment toward that end. Considering that the cost of power facilities in many major hydroelectric projects ranges from $50,000,000 to $200,000,000 or more, it is evident that millions of dollars may be at stake in achieving an economic balance within the basic design elements of power features alone. The study and analysis summarized in this paper demonstrate procedures used to optimize the net power benefits by balancing economic factors in design of the Lower Monumental Project, at mile 41.6 on Snake River, within the physical limits of site conditions, available head, stream flow, and various anticipated power system development and operational conditions. The power plant is planned for an ultimate six-unit full plant capability of 931,500 kw. The turbines will be Kaplan type with six adjustable blades. The runner will be 305 in. in diameter developing 212,400 hp under 93.5 ft of head. Special attention is given in this paper to the size and number of generating units and the correlation of turbine design characteristics to power plant costs and benefits. In effect, the analysis evaluates costs, benefits, and characteristics of sixty alternate power plants (thirty-two presented here) operating under three general system power development conditions and two assumed timing schedules of placing generating units in service. Construction of Lower Monumental

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

[a] Presented at the October 1960 ASCE Convention in Boston, Mass.

[1] Chf., Power and Electronic Data Processing Sect., U. S. Army Engr. Dist., Walla Walla, Corps of Engrs., Walla Walla, Wash.

Lock and Dam is being initiated in 1961 by the U. S. Army Engineer District, Walla Walla, Wash.

INTRODUCTION

Early in the history of hydroelectric power plant development, turbine efficiency received much attention as a major economic factor in power plant design. As a result, a high degree of perfection was achieved in the hydraulic efficiency and mechanical design of turbines in a comparably short period of time. Until the 1930's the demand for large blocks of power in most areas was limited and, for the most part, power plants were built to serve relatively

TABLE 1

Item	Lower Monumental Power Installation	
	Previously Proposed 1947 Site Selection Studies	Current Recommendation
General		
Project location, river mile	44.7	41.6
Normal pool elevation, ft-msl	533	540
Generator rated capacity, kw	58,000	135,000
Turbine rated capacity, hp	80,200	212,400
Runner throat diameter, inches	265	305
Elevation of centerline of distributor, ft-msl	437	403
Main unit spacing, ft	80	90
Initial Installation		
Units installed - number	4	3
Plant-rated capacity - kw	232,000	405,000
Ultimate Installation		
Units installed - number	5	6
Plant-rated capacity - kw	290,000	810,000

small power market areas or a specific industrial power need. Consequently, hydroelectric power plants were predominantly small in size. Due to the small potential savings associated with refined design in these small plants, considerable "rule of thumb" criteria were applied in formulating basic power plant physical features and in establishing turbine performance characteristics in relation to other economic factors. A major advance in the economic application of turbine design characteristics to specific project condtions was made with the advent of larger project and power systems during the development era of TVA, the Bonneville, Grand Coulee, and Hoover Dam projects, and numerous private developments from about 1925 to 1940. In general, the basic practices developed during that period prevailed throughout the United States until the post-war period. Marked changes have taken place in the relationship

of electrical and mechanical equipment costs, powerhouse structure, and excavation costs, of energy and capacity benefits, and economic factors related to financing. These changes, together with the advent of large installations, have afforded an opportunity for major savings in the basic design of power plants through detailed engineering studies to achieve an economic balance in meeting specific project conditions. Table 1 gives comparative data on the power plant proposed in 1947 for the Lower Monumental Project and for the plant currently (1960) recommended. The data illustrate the extent of changes in basic power plant design that are primary items of consideration in the analysis presented. The changes are quite typical of those which have taken place since 1948 in planning of Lower Columbia and Lower Snake River plants.

BASIC ELEMENTS IN POWER PLANT PLANNING

The determination of the scope and magnitude of power development in a hydroelectric project involves three basic steps: site location, height of dam, and magnitude of power installation. Although the presentation in this paper is confined to the determination of project power installation, the other two items are briefly outlined to indicate the interrelationship of all three.

Site Selection.—Unless there are otherwise controlling localized physical or geological features in the river reach of a proposed project, power plant cost and potential power output, as may be influenced by tailwater conditions, may be a significant factor in determining the most feasible site. In basin-wide plans the influence of other projects may be of more significance than local site conditions. This is particularly true in a series of run-of-river projects having the possibility of pool overlap. In such cases the pool level of the downstream project and location of the upstream project must be established simultaneously to achieve the most economic plan. Because site selection is normally the first basic determination to be made, it is usually necessary to assume the size and characteristics of the power plants involved. Although the assumptions made will influence results to some degree, normally the difference between the assumed plant and the final plant will affect results much less than other items being evaluated. This is particularly true if there is sufficient experience in establishing power installations for similar projects in the same area upon which to base judgment.

Height of Dam.—As previously noted, the determination of height of dam in a series of run-of-river projects will involve the siting of the upstream project. The cost of increasing height of dam will be balanced against gain in power benefit derived from reducing tailwater rise due to flood flows and power operations at the upstream project. Also, in projects where navigation facilities are included as on the lower Columbia and lower Snake Rivers (Fig. 1), reduction of excavation costs in navigation channels and lock approaches and reduction of stream velocities resulting from power peaking operations are important items.

Power Installation.—The magnitude of power installation in a hydroelectric development, once site location and height of dam have been selected, is essentially dependent on the cost of power facilities, the relative value of energy and capacity, and power development conditions; that is, the rate or amount of power output that is to be absorbed in the load. The economic balance between costs of excavation, power plant structures, turbines, and generators as related

to speed and setting involve millions of dollars. The principle physical elements associated with determination of power plant characteristics and the economic balance of costs and benefits are as follows:

1. Runner setting—speed relationship.
2. Monolith size in relation to wheel diameter.
3. Runner diameter—setting relationship.
4. Number of generating units.

In the case of Lower Monumental power plant (Fig. 2) these relationships were examined and analyzed more or less simultaneously. However, each is sus-

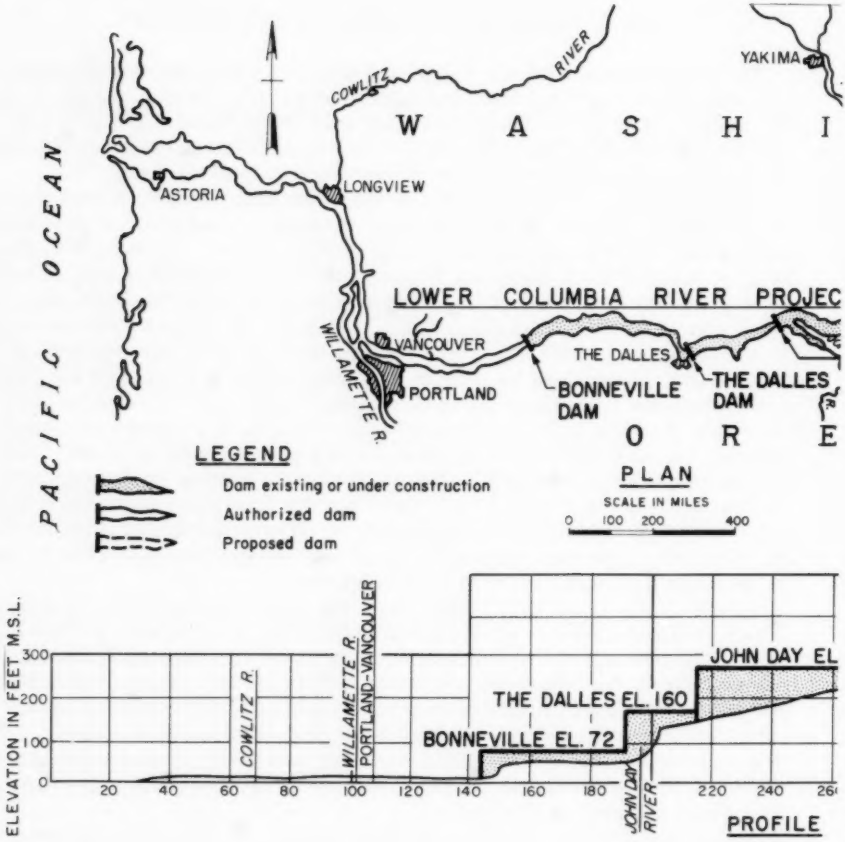


FIG. 1

ceptible to independent and step-by-step determination. Results are presented so that it is comparitively easy to see the relative effect of each relationship.

BASES FOR PROJECT POWER ANALYSIS AND EVALUATION

Project and System Operating Conditions.— (The conditions outlined are based on studies presented in Water Resources Development Report, Columbia River Basin, June, 1958.) The 50-yr economic life of the Lower Monumental Project is expected to span three general power system operational conditions.

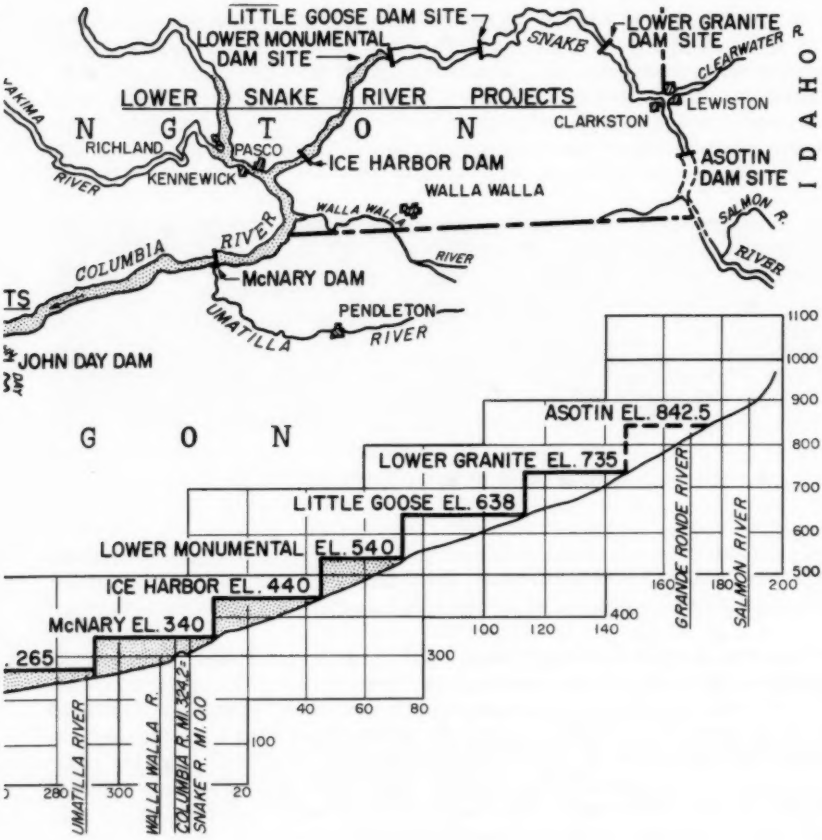


FIG. 1.—*CONTINUED*

The degree of Lower Monumental power development and relationship of power production to regional power needs and supply were based on three corresponding stages or levels of system water resource development with timing assumed as follows:

Initial System Development.—The initial point in reference to time and system development for power scoping studies has been assumed to be the presently scheduled completion date for the Lower Monumental initial project, December, 1967. It is estimated that storage available for power regulation in 1967-68 will aggregate about 13.3 million acre-ft, of which 1.0 million acre-ft is existent upstream from the Lower Monumental Project at the Idaho Power Brownlee Project (Fig. 1). Upper Snake River storage operated primarily in the interests of irrigation and flood control is not included. Regional power resources will be almost entirely hydroelectric and will serve peak requirements of some 16 million kw of firm power. During the following eight years to 1975, it is estimated that system power requirements will increase to about 23 million kw to be served by the addition of both hydro and thermal resources. Thermal resources are primarily required for energy generation under adverse hydro power production conditions. In this same period it is anticipated

FIG. 2.—LOWER MONUMENTAL LOCK AND DAM

that system upstream storage that is effective for power production will have developed to about 22.5 million acre-ft, including about 5.2 million acre-ft on Snake River and tributaries above Lower Monumental Dam and below Weiser, Idaho.

Intermediate System Development.—Between 1975 and 1985 it is expected that system peak power requirements will grow to about 33 million kw and probably will be served about one-third by hydro and two-thirds by thermal energy generation with respect to energy production under low stream flow conditions, and primarily by hydro insofar as peak capability requirements are concerned. It is assumed that during this 10-yr period system storage will expand to some 32.2 million acre-ft including about 10.2 million acre-ft on Snake River upstream from the Lower Monumental Project. Regional power requirements are anticipated to maintain a continuing growth which is expected to reach a magnitude of 47.5 million kw by 1995, the assumed end point of the intermediate period of project economic life. Power resource surveys indicate that after about 1975, or possibly several years earlier, a substantial amount of low-cost hydro installation may be economically installed over and above critical period firm power requirements on the basis of thermal energy re-

placement value. These surveys also indicate that after 1985 regional needs for peak power will exceed the capability of thermal power and hydro plants installed primarily on the basis of their energy-producing ability, thus making the addition of less costly hydro installation economically feasible for peaking purposes.

Ultimate System Development.—In the studies performed, expansion of system upstream storage beyond a total of 32.2 million acre-ft has not been considered. Regional power requirements are forecasted in excess of 100 million kw by the year 2017, which is the end of a theoretical 50-yr economic life period assumed for the Lower Monumental Project for economic evaluation purposes. It is expected that by the year 1995 and no later than 2005, unless unforeseen elements are introduced in the future to change the forecasted power load and resource pattern of development drastically, all feasible hydro installations will be needed to meet regional peak power demands.

Stream Flow and Period of Record.—The 20-yr minimum flow period of record, July 1928-1948, was used in power plant outlook studies to represent the average for the 50-yr exonomic life. Regulated stream flows for the 1928-48 period are those resulting from general system monthly regulation studies with the plant operating as an integral part of Pacific Northwest water resource development.

Pool Elevations.—Normal and minimum controlled pool elevations for the Lower Monumental Project will be El. 540 and El. 537, respectively. These pool levels, which were established in conjunction with site selection for Little Goose, the next project upstream, will provide optimum combined power production relationships for these two projects with due consideration given to progressive development of upstream resources and increased demand for power capability through the succeeding 50-yr on the same basis as used to determine power installation. With the project having no assigned flood control regulation responsibilities, pool fluctuations will be primarily in the interests of meeting project power peaking requirements. The planned pool levels are fully compatible with operational needs for slack-water navigation. Studies indicate that approximately 20,000 acre-ft of pondage volume provides sufficient flexibility for regulation of reservoir inflows to meet system power production demands.

Cost of Alternative Power.— As an economic measure of the scope or magnitude of power development feasible at Lower Monumental, values equivalent to the cost of the cheapest alternative power source have been used. These values are the average of non-federal, publicly-financed steam costs within the Pacific Northwest region as estimated by the Federal Power Commission and as used in th Water Development Report of June, 1958.

Energy.—Both firm and secondary or steam-replacement energy evaluations are based on an at-site value of 3.2 mills per kw hr.

Capacity.—Dependable capacity evaluations are based on an at-site value of $11.28 per kw yr.

The preceding costs are primarily associated with the intermediate and ultimate levels of system development. The costs of other hydroelectric power development which could be available as an alternative source during initial system development have not been used. The early system condition will probably exist less than 10-yr following completion of Lower Monumental initial construction and has minor significance as related to the outlook for

the Lower Monumental power plant, except perhaps as to the number of units initially desirable.

POWER PLANT INSTALLATION ANALYSIS

Power Plants Considered.—In all, 60 different power plants were examined. Five turbine sizes were selected (274 in., 281 in., 288 in., 295 in., and 305 in. wheel diameters) to cover the range of economic and physical practicability (thirty-two power plants covering four wheel sizes, 281 in., 288 in., 295 in., and 305 in., are presented here); four settings, at El. 394, El. 399, El. 404, and El. 414 ft above mean sea level, compatible to examination of both maximum energy and maximum capability for the five turbine sizes were chosen. Five unit, six unit, and seven unit ultimate plants, and in the case of the smallest turbine wheel size, an eight-unit plant, were assumed for coverage of the practical and economic range of power installation. For convenience and ease of reference, wheel size, setting, and number of units in the ultimate plant are used in combination to identify each power plant scheme. The turbines were designed to develop rated horsepower at critical head corresponding to the condition of minimum headwater pool and tailwater with ultimate number of units at full gate discharge and as limited by cavitation using a 3-ft head above vaporization as a margin of safety. Turbine prototype characteristics were based on Kaplan turbine (Ice Harbor model) unit performance characteristics, as shown in Fig. 3. Gererators, assumed to operate at 0.95 power factor at 115% rated capacity, were matched to rated turbine output. Total plant capabilities covering a variation of about 450,000 kw, the minimum installation considered being approximately 580,000 kw and the largest about 1,035,000 kw, were examined. The dimensions of imbedded turbine parts, as well as unit spacing and length and depth of draft tubes, were based on experience factors obtained from existing Kaplan installations. Monolith spacing was examined through a range of scroll-case-width to wheel-diameter-ratios of 2.9 to 3.2. A factor of $3.0 \times D_{th} + 13$ ft for pier width was finally used in determining monolith spacing of installation comparisons.

Installation Schedule.—The scheduling of installation-in-service and the basis for crediting capacity and energy benefits are shown in Table 2. Two rates of installing capacity were analyzed, one assuming all capacity in service and fully usable by the year 1995, and the other with capacity installed at a slower rate following installation of the fifth unit and assumed usability in the load credited more conservatively.

Project Costs.—Construction costs of project features other than power facilities remained constant for all comparisons except for the right abutment embankment, shortened or lengthened as required by the various powerhouse lengths. Power plant construction costs were developed for each power plant considered.

Annual costs include interest and amortization, operation and maintenance, and major replacements. Fifty-year weighted average annual costs were prepared on the premise that all project investment costs would be amortized within the 50-yr economic life of the initial project.

Energy Determinations.—Basic potential energy determinations were made using a normal pool elevation of 540 ft msl. Energy output in all cases was adjusted to account for energy loss incurred in load factoring operations. The

alternative plants were assumed to handle equal volumes of water through equivalent over-all time periods. Load factoring cycles varied through the 50-yr project life from 67% for average annual operation to 85% of total time in critical periods. Relative energy differentials resulted essentially from efficiency differences between power plant characteristics.

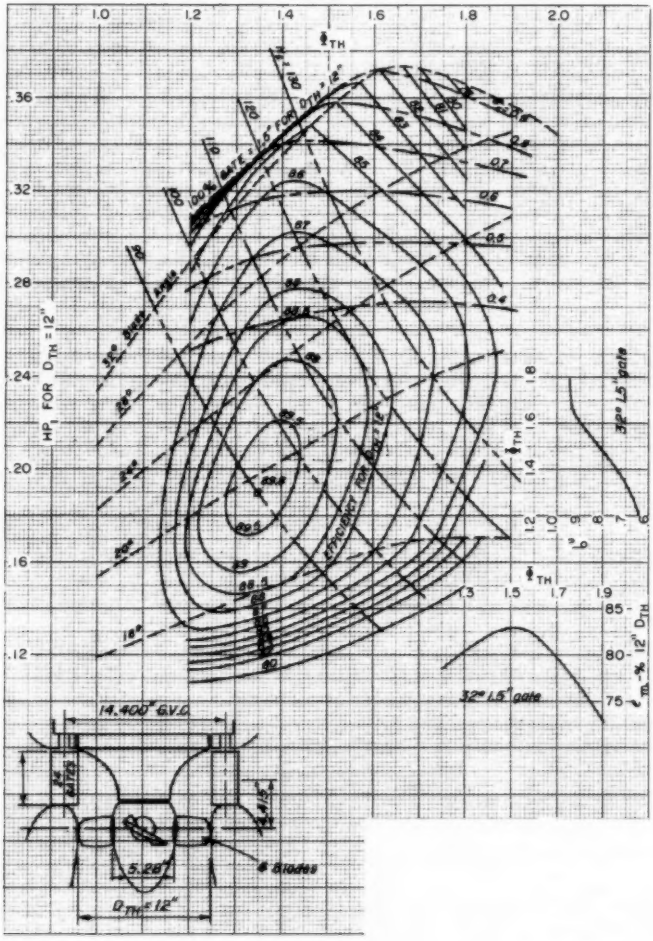


FIG. 3.—PERFORMANCE CHARACTERISTICS KAPLAN TURBINE ONE-FOOT HEAD

Power Benefits.—The basis for evaluation of creditable capacity and energy generation of each power plant alternate is shown in Table 2. Initially, credit is taken for primary energy only, and capacity credit is restricted to a load factor of 100% based on average critical period generation. It is assumed that

TABLE 2.—POWER PLANT INSTALLATION SCHEDULE AND BASIS FOR EVALUATION[a]

Time Period	Length of Period-Years	Installation of Units		Energy		Capacity	
		Number	Year Installed	Units	Credit	Units	Credit
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Schedule #1 & #2							
First period, average of 1967-1975 Spot Values	8	3	1967				
1967 Spot Value				3	7 month avg prime	3	7 month avg prime
1975 Spot Value				3	9 month avg prime	3	9 month avg prime ÷ .71 (System load factor)
Second period, average of 1975-1985 Spot Values	10	4th	1975				
1975 Spot Value				4	9 month avg prime	4	9 month avg prime ÷ .71 (System load factor)
1985 Spot Value				4	Avg annual energy	4	Full overload
Third period, average of 1985-1995 Spot Value	10	5th	1985				
1985 Spot Value				5	Avg annual energy	4	Full overload
1995 Spot Value				5	Avg annual energy	5	Full overload
Schedule #1, 5 UNIT ULTIMATE PLANT							
Fourth period, 1995-2017	22	5	1985	5	Avg annual energy	5	Full overload
Schedule #1, 6 UNIT ULTIMATE PLANT							
Fourth period, average of 1995-2005 Spot Values	10	6th	1995				
1995 Spot Value				6	Avg annual energy	5	Full overload
2005 Spot Value				6	Avg annual energy	6	Full overload
Fifth period, 2005-2017	12	6	1995	6	Avg annual energy	6	Full overload
Schedule #2							
Fourth period, 5 unit ultimate plant, 1995-2017	22	5	1995	5	Avg annual energy	5	Full overload
Fourth period, 6 unit ultimate plant, 1995-2017	22	6th	1995	6	Avg annual energy	6	Full overload

Note: First three periods common to all turbine wheel diameters, settings and ultimate plants.

[a] Project 50-yr Economic Life (1967-2017)

by 1975 capacity may be utilized at a system critical period load factor 71% and that by 1985 the full capability and secondary energy developed by four units can be absorbed in the load. Through the balance of initial project economic life, 1985-2017, it is assumed that secondary energy can be utilized for replacement of more costly steam generation as soon as additional units are installed. However, the capability of the fifth unit, the last unit in the minimum or basic power plant of each wheel size and setting series, is assumed not to be fully usable nor to have full dependable capacity value until 1995. Beyond 1995, the rate of installation and crediting of capacity benefit is examined on two bases. The first and slower schedule, designated as schedule 1, assumes unit 6 installed in 1995 but not fully usable in the load until 2005. The second schedule assumed all units installed and fully creditable in 1995.

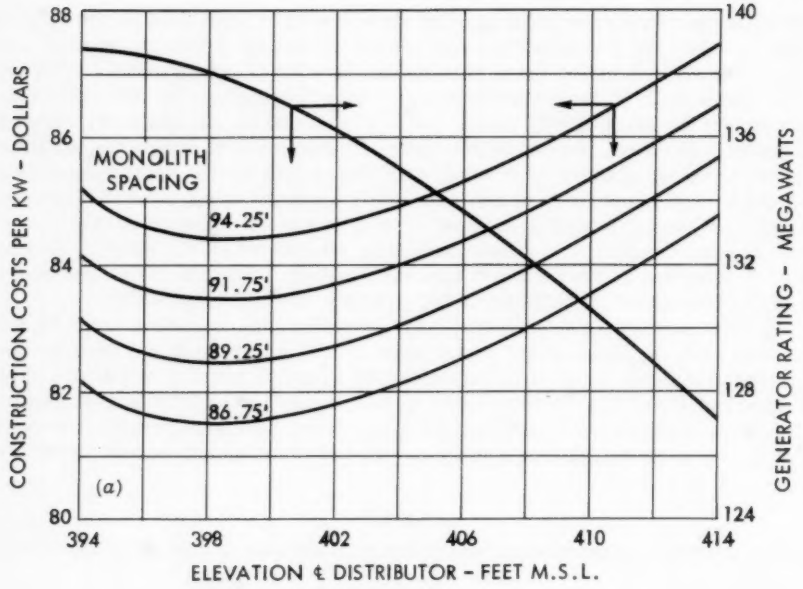
Runner Setting - Speed Relationship.—The results of studies made in connection with establishing the power plant for John Day Lock and Dam, located on the Columbia River approximately 90 miles below the confluence of the Snake and Columbia Rivers, were applied in the Lower Monumental studies. The optimum speed for a generating unit at a given setting is primarily dependent on the relative value of energy and capacity in conjunction with associated

TABLE 3

Elevation of centerline of Distributor, in feet MSL	Wheel Diameter, in Inches			
	281	288	295	305
	Synchronous Speed, in revolutions per minute			
394	102.9	102.9	102.9	97.3
399	100.0	97.3	94.7	92.3
404	94.7	92.3	92.3	87.8
414	85.7	83.7	81.8	80.0

turbine, generator, and structural costs. A unit designed for maximum energy production and high load factor operation will economically favor a slower speed than a peaking unit designed for low load factor operation. In evaluation, the change in cost of main unit generators and associated electrical equipment for each change in speed is balanced against the gain or loss in energy and capacity benefits. If maximum energy is the criterion, the cost of equipment is balanced against gain in turbine efficiency. If maximum capability at least cost is desired, the cost of equipment p,us the loss of efficiency is balanced against realizing the benefit of full power potential in the turbine within the limits of critical runner sigma.

In the general case, the best speed will fall in between the extremes. An approximation of the position the power plant is to take in the load through its economic life is required to arrive at the economic speed choice. In the early years of Lower Monumental Project's life, capacity beyond that to develop firm power will have little value. Twenty-five years thereafter, the Pacific Northwest power system will require considerable hydro peaking power in support of thermal plants. With this expectancy, it was found that optimum unit synchronous speed for each unit size and setting would be the maximum speed required to develop full turbine horsepower at a critical head resulting from full pondage drawdown and tailwater with ultimate plant rated discharge.



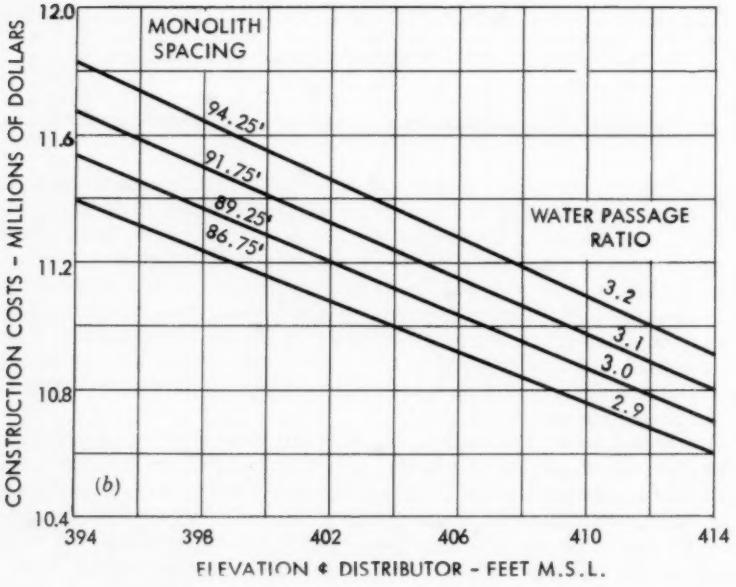


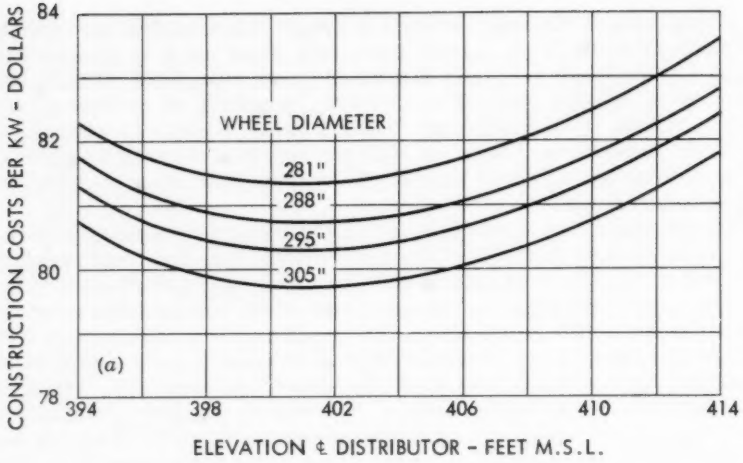
FIG. 4.—RATIO OF WIDTH OF WATER PASSAGE TO WHEEL DIAMETER IN RELATION TO DEPTH OF SETTING - 305" KAPLAN TURBINE

As an indication of the relative worth of proper speed choice, the capitalized net benefit for the six-unit Lower Monumental Plant using an interest rate of 2-1/2% and a 50-yr economic life would range from $680,000 ($24,000 net benefit) to $1,400,000 ($50,000 net benefit), depending on whether energy or capacity benefit is the objective. Table 3 shows comparative synchronous speeds for Lower Monumental. It will be noted that some of the speeds are not ideal from an electrical standpoint; however, they were used to develop comparative cost data.

Monolith Size.—The determination of monolith size or spacing of units, the water requirements being fixed for a given wheel size and rated capacity, is dependent on the configuration of the spiral case. Assuming losses to be the same for each configuration, the evaluation of varying monolith spacing becomes purely a cost comparison. The results of evaluations made for one main-unit bay for Lower Monumental to determine the relationship of costs to monolith spacing, using a variable ratio of water passage width (scroll case) to runner diameter, are shown in Fig. 4. As might be suspected, with hydraulic losses assumed fixed, the cost differential per increment of change in monolith width remains relatively constant. A savings of $54,000 per ft of width is indicated which is 1.0% to 1.2% of the total structural costs. Reducing monolith spacing from a theoretical 94 ft to 90 ft with a 305-in runner and a six-unit plant represents a saving of approximately $1,300,000. Due to the unknowns associated with hydraulic losses in the scroll case it was not considered practical to reduce the wheel-to-scroll-case-width ratio to less than 3.0. This, with slight variance due to rounding off to even feet, was used in wheel-diameter, speed-setting studies. Additional research in evaluation of the effects on turbine performance associated with changing the configuration of the turbine scroll case could reveal significant additional potential savings in reducing monolith widths.

Runner Diameter - Setting Relationship.—After determination of the criterion for best synchronous speed and unit monolith spacing as related to runner diameter, the economic relationship of runner diameter to setting may be evaluated. Selection of the optimum setting involves all physical, operational, and economic factors of the basic elements in design of the power plant. Fig. 5 shows that within a wide range the construction costs vary directly at essentially a uniform rate as the runner setting is lowered. Costs increase with an increase in runner diameter in a similar manner. It should be noted, however, that the cost per kilowatt is highly influenced by the setting. It is of considerable interest that the indicated lowest cost per kilowatt is achieved with the center line of the distributor set approximately 40 ft below tailwater elevation at critical head, which is appreciably lower than has been normal practice. Fig. 6 shows the ralationship of turbine setting to plant capability for five- and six-unit installations and the various runner diameters demonstrating the increased capability associated with the lowering of units. Fig. 7 is an envelope curve showing the energy generation for the same installations. The energy picture shown is the average annual generation over the 50-yr project economic life. Although not illustrated on the basis of prime energy only, during the early period of project life optimum generation would be achieved with the center line of distributor set at about El. 420.

The relationships of annual costs to turbine setting for the various power plants are given in Fig. 8. These cost data, in conjunction with the performance



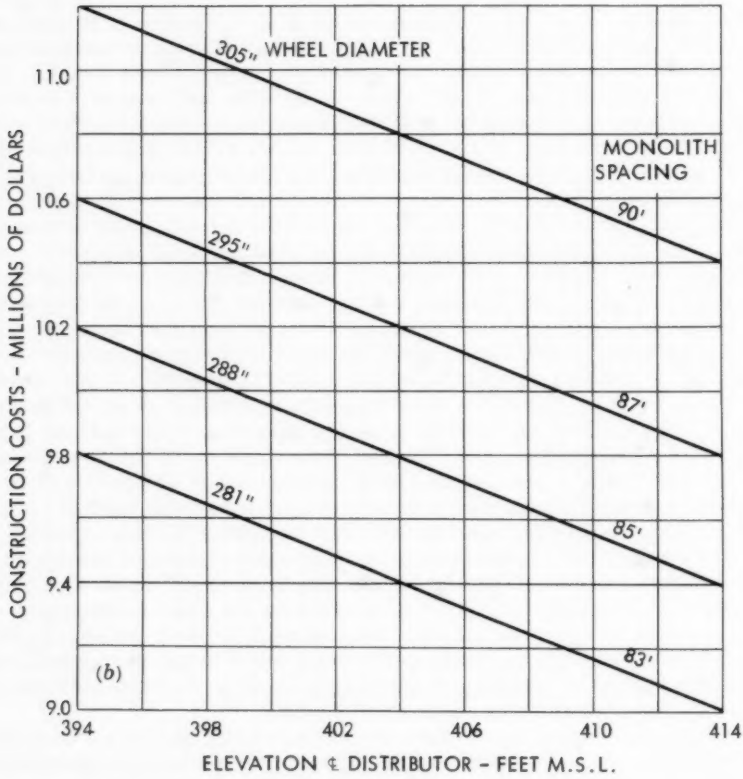


FIG. 5.—COST OF ONE MAIN UNIT BAY IN RELATION TO DEPTH OF SETTING

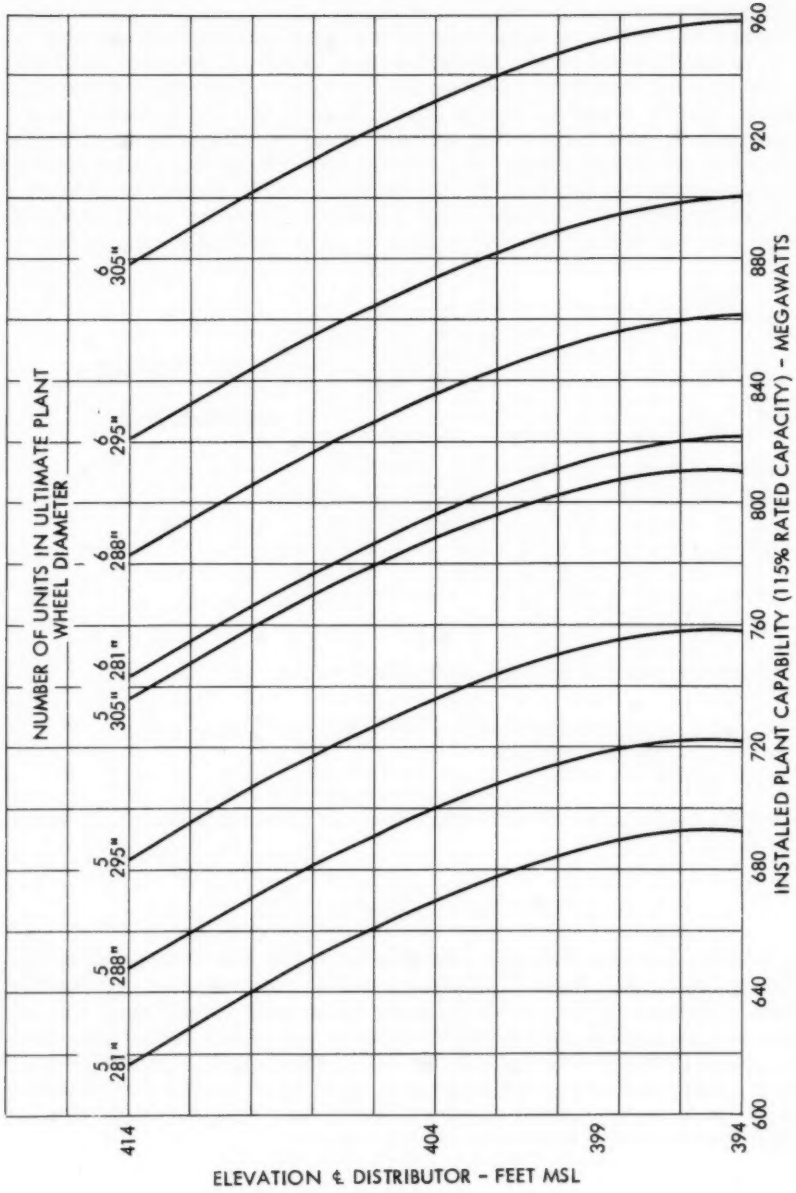


FIG. 6.—RELATIONSHIP OF DEPTH OF SETTING AND INSTALLED PLANT CAPABILITY

characteristic data illustrated by Fig. 6 and 7, provide the basic information necessary for economic evaluation of the alternate plants. By application of established power values for capacity and energy to the power potential of each plant considered, the gross power benefits associated with the alternate schedules of installation can be readily determined. The gross power benefits for five- and six-unit installations of the various runner diameters and settings are shown in Fig. 9 and 10 for installation shcedules 1 and 2, respectively.

These graphs demonstrate that optimum gross benefits are achieved with a distributor centerline setting at about El. 400, indicating the predominant influence of gross capacity benefits in relation to energy benefits on the optimum turbine setting. Similar conditions would be expected for any plant which would be credited with significant capacity benefits, especially plants at which

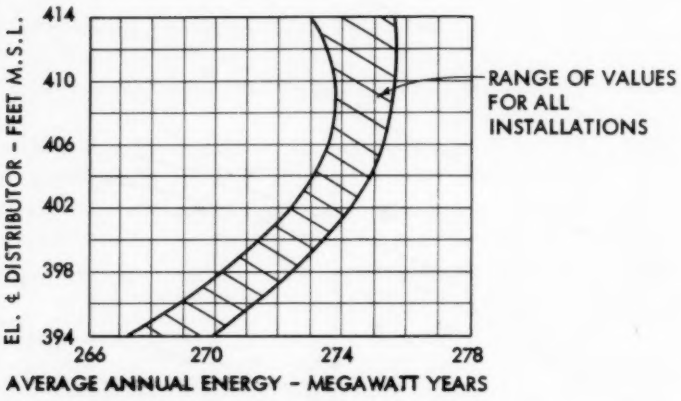


FIG. 7.—RELATIONSHIP OF DEPTH OF SETTING AND AVERAGE ANNUAL ENERGY

capacity is installed over and above that which can be justified on energy benefits alone.

SELECTION OF POWER UNITS

By combining the cost determinations shown in Fig. 8 with the gross power benefits shown in Fig. 9 and 10, summary curves demonstrating the relative economic feasibility of the plants evaluated can be readily developed. Fig. 11 summarizes the results of all thirty-two plants for installation schedules 1 and 2. The graphs give net power benefits for all plants studied as related to total installed plant capability. Similar graphs could be developed for any other assumed installation schedule. For any given condition or total power installation, the most feasible units can be readily selected.

CONCLUSIONS

The study precedures presented have been successfully applied in determining unit size and characteristics and initial and ultimate installations for the

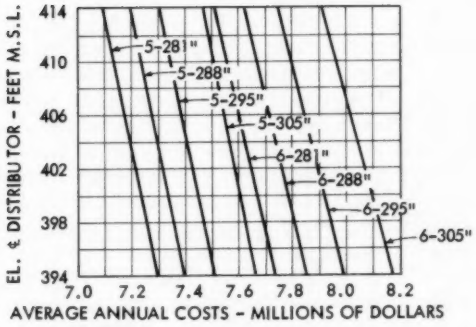


FIG. 8.—RELATIONSHIP OF DEPTH OF SETTING AND AVERAGE ANNUAL COSTS

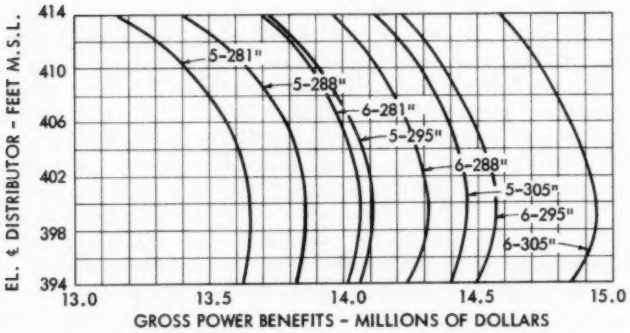


FIG. 9.—RELATIONSHIP OF DEPTH OF SETTING AND GROSS POWER BENEFITS, SCHEDULE #1

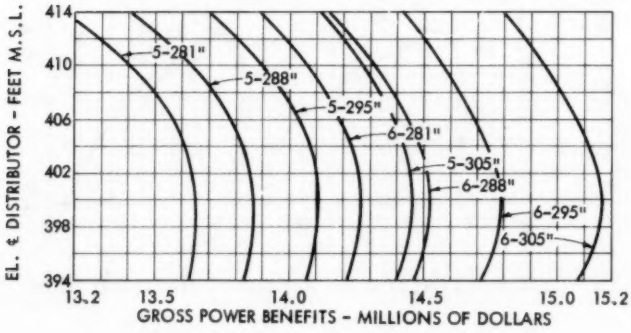


FIG. 10.—RELATIONSHIP OF DEPTH OF SETTING AND GROSS POWER BENEFITS, SCHEDULE #2

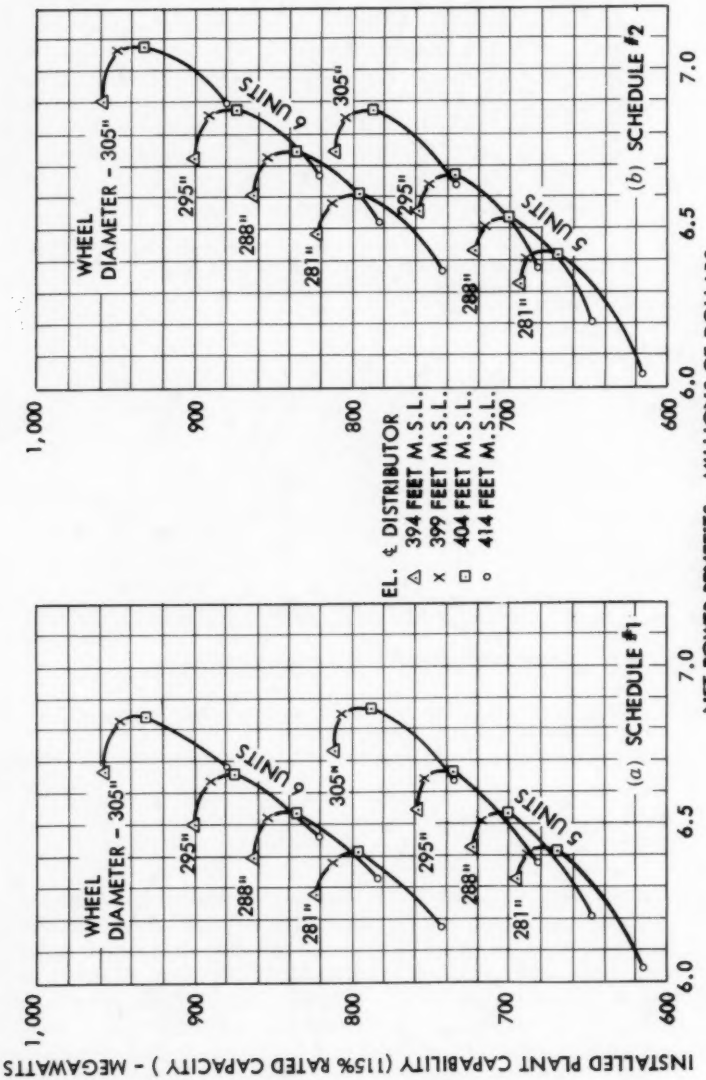


FIG. 11.—ECONOMIC INSTALLATION IN TERMS OF PLANT CAPABILITY

Ice Harbor, John Day, and Lower Monumental power plants. These power plants will have a total capability of about 4,500,000 kw. The procedures were first applied in the analysis of the Ice Harbor Project by manual short-cut methods. The John Day and Lower Monumental studies to a large extent used an electronic computer which no doubt has improved the results. The analysis presented demonstrates study procedures and results which have been invaluable in the analysis of major power projects, but should not be construed to eliminate the need or importance of proper selection of basic criteria which must be established for the analysis.

It should also be noted that many factors related to multipurpose use, the characteristics of the power system as well as adjacent plants on the stream, may be of utmost significance to final determinations of initial and ultimate installations. In the case of the Lower Monumental power plant, siting was influenced by navigation conditions and the pondage provided was influenced by more favorable pondage at the proposed adjacent upstream Little Goose plant. An early life period of system surplus in hydro plant capability associated with installations to achieve prime energy and concern as to the feasibility of providing units for capacity value only which would result in plant operation under a 30% load factor were also primary considerations. Although the studies indicated that runners larger than 305 in. in diameter would prove more economical, the horsepower rating is over 50% greater than any Kaplan unit built and tested as of 1961. At Lower Monumental the large water capacity of 23,000 cfs for one unit in relation to minimum discharges during peaking operations is of significance. With consideration of all these factors in conjunction with the detailed studies to achieve an economic balance in design within the power plant, six units having a runner diameter of 305 in. at optimum economic setting within a 90-ft monolith were selected.

The studies presented demonstrate a practical method of establishing a correlation of costs and benefits of various project characteristics toward reducing the areas and extent to which professional engineering judgment must be applied. However, the economic significance of the items evaluated also bring to sharp focus the importance and need for a highly competent technical staff for proper application of the specific evaluations and similar items largely dependent on judgment in establishing basic plans for a hydroelectric power project.

ACKNOWLEDGMENTS

Power plant installation studies for the Lower Monumental Project were made by the Power Section Staff, Planning and Reports Branch, Walla Walla District, Corps of Engineers. Power plant cost criteria were prepared by the Hydro-Electric Design Branch, North Pacific Division, Corps of Engineers. Computer programs used in determining basic power output data and power plant cost data were developed by Chester T. Scott, Mechanical Engineer, of that staff. System water regulation studies from which power output data was derived and computer programs used in power plant load factoring studies were accomplished by the Power Section, Water Control Branch, North Pacific Division, Corps of Engineers. The Baldwin-Lima-Hamilton Corporation and Allis-Chalmers Company provided considerable turbine data and basic turbine criteria. The assistance of Glenn H. Von Gunten, F. ASCE, Chief, Planning and Reports Branch, in the preparation of this paper, is gratefully acknowledged.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

POWER FROM THE SEA

By El Sayed Mohamed Hassan[1]

SYNOPSIS

Evaporation exceeds precipitation over the Red Sea by 1.5 x 10^6 cfs. All this amount is now supplied from the Indian Ocean through the narrow Straits of Bab El Mandeb. If these straits are dammed, the loss of water will result in lowering the Red Sea level by 12 ft per yr. A scheme is presented to utilize this drop to generate electricity at a very cheap cost.

INTRODUCTION

Among the great engineering projects contemplated, a proposal for what is probably the most ambitious of all will be presented herein. This is an oceanic hydroelectric power plant that dwarfs all existing plants and if constructed, will have far-reaching effect on Africa and the world.

THEORY

Evaporation exceeds precipitation[2] over the Red Sea (Fig. 1) by about 1.5 x 10^6 cfs. This amount is now replenished from the Indian Ocean through the Straits of Bab El Mandeb. If these Straits and the Suez Canal are dammed, this evaporation will lower the level of the Red Sea by about 12 ft per yr. By

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

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[2] "The Oceans, Their Physics, Chemistry and General Biology," by H. U. Sverdrup, M. W. Johnson, and R. H. Fleming, Prentice Hall, Inc., Englewood Cliffs, N. J., 1942.

allowing a part of the evaporated water to be replaced, electricity can be generated. The difference in level between the Red Sea and the Indian Ocean will continue to increase as long as only part of the evaporated water is replaced. This can go on until the head reaches a desired amount. The maximum

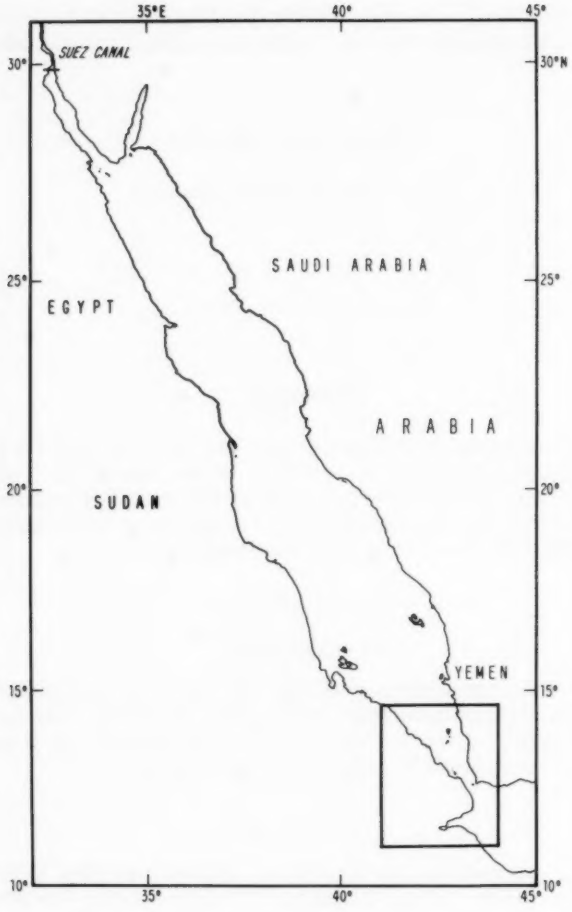

FIG. 1.—THE RED SEA

head depends on the site of the dam, but it need not be less than 500 ft. The maximum amount of electricity that can be produced with such a head is 70,000,000 kw. Other considerations will reduce this to about 50,000,000 kw, as will be seen later. This is about 1/5 of the electric energy generated in all

the world in 1959, or more than the total energy from all sources used in the United Kingdom in 1948.

SITES FOR THE DAM

The geography and topography of the Bab El Mandeb Straits make the choice of a site for the dam arbitrary within a large area since the sill is situated inside the Red Sea, north of the Straits. Thus the narrowest section of the Straits is also the deepest, while the shallowest depth occurs in a section where the Straits are wider. The volume of the dam will be approximately the same if constructed anywhere between these two extremes. Other factors, such as the nature of the sea bottom, the availability of suitable materials, or the engineering difficulties, will have to be considered to choose one location from all the possible ones. Three sections of the Straits of Bab El Mandeb are shown in Figs. 2 and 3. Section AA' is the narrowest part where the width is about 12.5 miles, and the greatest depth is about 950 ft. Section BB' is 21 miles wide, but the maximum depth is about 600 ft. Further north the Straits become relatively shallow, and the maximum depth is about 500 ft, but the width becomes about 60 miles. The difficulties of damming the Straits at any of these places are certainly great, but they are not insurmountable.

GENERAL

The amount of evaporation is roughly proportional to the area of the sea. If the sea level is to be lowered by 600 ft, its area and consequently the evaporation will be reduced by one third of the present amount. This will occur 50 yr after damming the entrance if no water is allowed to come into the sea. If water flow is allowed to compensate for part of the evaporation, it will take longer for these conditions to be attained.

The effect of the increase of salinity on evaporation is small. If no water is allowed into the sea, the salinity will be doubled after 200 yr. This will decrease the evaporation by about 3%. The combined effect of decrease of area and increase of salinity will reduce the possible maximum of generated electricity from 70,000,000 kw to 50,000,000 kw for a 500-ft head.

A great variety of schemes is possible that would generate electricity at an attractive cost. The amount of electricity generated can be tailored to the demands through the control of the amount of water inflow to the Red Sea. As long as the inflow is less than the water evaporated, the generated electricity by a certain amount of water will continue to increase. One scheme will be computed to illustrate these points.

ASSUMPTIONS

1. Two dams will be built, one at the southern end of the Red Sea and another at the southern entrance of the Suez Canal. The southern dam will be built of rock, sand, and clay of local origin at a cost of $0.80 per cu yd. It will have a slope of 4:1 towards the Indian Ocean and a 3:1 towards the Red Sea. It will rise 30 ft above the sea level and be 30 ft wide at the top (Fig. 4). With these specifications the southern dam will cost a little less than $2,000,000,000.

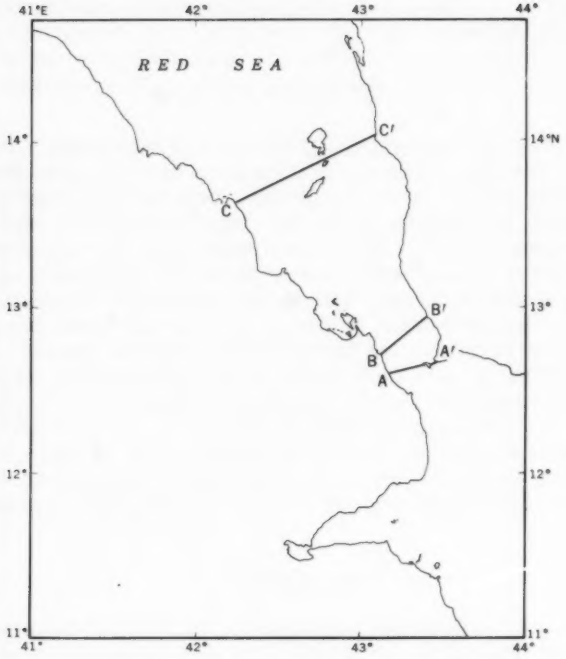

FIG. 2.—THE STRAITS OF BAB EL MANDAB, SHOWING POSITIONS OF THE CROSS SECTIONS FOR WHICH THE PROFILES ARE GIVEN IN FIG. 3

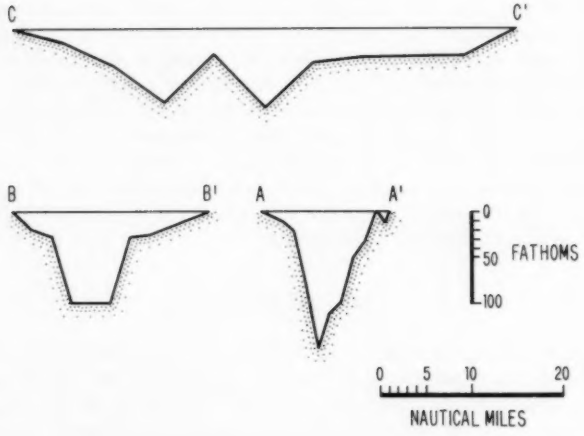

FIG. 3.—PROFILES OF THE THREE CROSS SECTIONS SHOWN IN FIG. 2

The cost of the northern dam will be insignificant in comparison. The combined cost of construction will be taken therefore as $2,000,000,000.

2. The electric generating system will generate 75% of the maximum available power as computed from the water head. It will cost $200 per kw. It will need replacement of parts to the extent of 4% of its capital cost every year.

3. After the completion of the dams, 2 yr will pass without allowing any water from the Indian Ocean into the Red Sea except as necessary through navigational locks. After that, and throughout the period of this computation, half as much water as evaporates will be allowed into the sea. As a result of this the sea level will drop by 24 ft in the first 2 yr, and 6 ft per year from that time onwards.

4. After the 25th yr of operation, another dam will be built in the southern end of the Gulf of Suez. This dam will cost $1,000,000,000 and will be finished in 10 yr.

5. The rate of evaporation will continue to be the same throughout the period of the computation. This assumption simplifies the computation, although it is strictly not valid as evaporation will decrease with decrease of

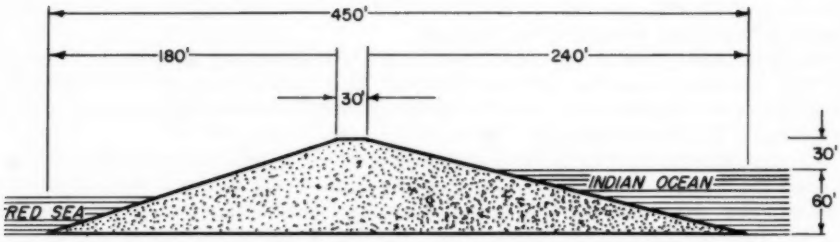

FIG. 4.—PROFILE OF THE PROPOSED DAM AT A PLACE 60 FT DEEP

surface area. However, as long as the power generated is within the maximum possible limit, computations resulting from this assumption can still be realized.

6. The necessary loan to carry out the project will be available at 4% interest compounded annually.

REQUIREMENT

The price of electricity at the source should be 3.5 mills per kwh. In comparison, the price of electricity generated from atomic plants is predicted by the Atomic Energy Commission to be 6.57 mills per kwh in 1975. Also, any power plants producing electricity at 5 mills per kwh in the United States could capture 43% of the market, and any plant producing it at 4 mills per kwh could capture 71%.

RESULTS

A computation carried out according to the preceding assumptions and requirement gave the following results:

1. The cost of new equipment that has to be bought annually is 58,500,000.
2. The cost of renewing existing equipment will thus rise annually by $2,400,000.
3. The annual increase of income due to increased power is $8,900,000.
4. After 6 yr from the start of generating electricity the income will be sufficient to meet the annual cost of equipment and replacements.
5. After 46 yr, the debt reaches a maximum of $6,717,000,000.
6. By the 82nd yr the entire debt, with its interest charges, is liquidated.
7. The net revenue after the 82nd yr is about $500,000,000.

CHARACTERISTICS

This project has some new features that make it unique among existing hydroelectric projects. Because of their novelty they are enumerated here to bring them into sharper focus.

1. It makes use of direct evaporation due to solar energy.
2. Power is produced by lowering the level of the lower reservoir rather than raising the level of the higher one.
3. Only water that is needed for power generation needs to be allowed in the sea. There is no need to allow water flow for flood control as the water level in the upper reservoir is constant.
4. The power that is not produced is actually stored, and when it is needed, it will become available and with interest. The interest comes from the fact that due to the increase of head, the same amount of water will generate more electricity at a later time.
5. Because of the increasing salinity in the Red Sea, it can eventually be mined for different chemicals, and thus the project will be producing some of the raw materials that the new industries will need.
6. The problem of silting which plagues many river projects is non-existent.
7. Because of the ever increasing head (at least during the first 50 yr of operation) special turbines will have to be designed to work efficiently over a wide range of heads.
8. The dam will be protected from the action of large waves because of the shape of the Gulf of Aden and the Bab El Mandeb Straits.
9. The project also offers the intriguing possibility of the first planned tapping of the nutrients stored in the deep water of the oceans. By restricting the water intake to certain depths in the Indian Ocean, it is possible to pump into the Red Sea more than 1,000,000 tons of fish nutrients per year. This could develop a major fishing industry in that area. Many problems have to be investigated, however, before such an extension of the project is made.[3]

PROBLEMS

Problems will, of course, arise. The most important are as follows:

1. Shipping which now passes through the Red Sea without any locks will have to be carried through systems of locks. These should be of adequate size to handle at least the volume of traffic that can pass through the Suez Canal.

[3] "Food from the Sea," by E. M. Hassan, Bulletin of the Oceanographic Inst., Univ. of Cairo, United Arab Republic, 1961 (in press).

2. Existing port facilities will be rendered useless, and new ports will have to be constructed as the sea level drops. This is a major expense to the countries concerned, but may well be a profitable investment considering the benefits expected to the whole area from industrialization.

GENERALIZATION

The same condition on which this project rests also exists in other marginal seas. This condition is the inherently unbalanced water budget of these seas. The deficit or the excess is taken care of by transporting water over the sill from or to the nearest ocean. Seas with a surplus budget are not suitable for the generation of electricity since, to produce a head there, the sea level has to be raised substantially. This will flood dry land which is very undesirable.

Example of seas with a deficit water budget are the Mediterranean Sea and the Persian Gulf. Other reasons make them not as suitable as the Red Sea for the generation of electricity.

CONCLUSIONS

1. Because of excess evaporation, the Red Sea will drop by about 12 ft per yr, if dammed.

2. A dam with the necessary installations for generating electricity and systems of locks for shipping will cost something like $33,000,000,000. The actual debt needed is less than $7,000,000,000, the rest being met by the revenue.

3. The system can be paid for in 82 yr while producing electricity at 3.5 mills per kwh. A conservative estimate of the amount of electricity possible to generate is that it will be more than 15 times that generated by the Grand Coulee Dam.

4. The advantages of getting that much electricity for industrialization greatly outweighs any foreseeable disadvantages.

ACKNOWLEDGMENTS

The writer wishes to thank Charles S. Cox and Robert S. Arthur of the Scripps Institution of Oceanography for many stimulating discussions and valuable suggestions, and David M. Garner of New York University for reading the manuscript.

Since the writing of this article, it has been brought to the writer's attention that the principle of this project has been suggested by M. R. Bigarre[4] in 1940. The writer has not been able to locate the original reference.

4 "Engineers' Dreams," by W. Ley, The Viking Press, New York, N. Y., 1954.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

OROVILLE DAM AND APPURTENANT FEATURES

By W. G. Schulz,[1] D. P. Thayer,[2] and J. J. Doody,[3] Fellows, ASCE

SYNOPSIS

This paper describes the investigations and design studies leading to the selection of a fill type dam for construction at the Oroville site. This 735-ft high dam will be the highest dam in the United States (as of its completion date) and the highest fill type dam in the world. Another novel aspect of the project is the two-state pumping lift of the pump storage plan that results in the water passing from the main reservoir through the three smaller reservoirs downstream during the generating cycle and then reverses itself back through the four reservoirs during the pumping cycle of the two power plants involved.

DESCRIPTION OF PROJECT

The Oroville Dam of the Feather River Project is a major unit of "The California Water Plan" of the California Department of Water Resources. The water plan endorsed by the State Legislature is a master plan to guide and coordinate the planning and construction by all agencies of works required for the control, protection, conservation, and distribution of California's water resources.

Oroville Dam, located near the town of Oroville in Butte County, will be a fill type dam about 735 ft high containing approximately 80 million cu yd of fill. It will be the highest dam in the United States and the highest fill-type dam in

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

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the world. The 3,523,000 acre-ft reservoir will be of the multipurpose type and will function for flood control, conservation, and power generation. (Fig. 1.) In the left abutment of the dam there will be a 600 megawatt underground power plant, and on the right abutment will be a combined spillway and outlet works structure.

Downstream from the Oroville main dam there will be three smaller dams, a power canal, a second power plant, and probably a fish hatchery. The first of these smaller dams is known as the Thermalito Diversion Dam and it serves to divert the flow from the main river channel into the Thermalito power canal

FIG. 1.—OROVILLE PROJECT

that, in turn, discharges into the Thermalito forebay, created by the second of the smaller dams. The flow then passes through the power drop at the second power plant called the Thermalito power plant and via a tailrace channel into the Thermalito afterbay that is contained by the third of the smaller dams. After making the necessary irrigation releases into the Western canal and the Sutter-Butte canal, the reregulated flow is returned to the main river at the river outlet. The Thermalito plant will have 105 megawatt capacity, part of which will be provided by pump turbines, as will be the case with part of the capacity of the main Oroville plant. During the pumping part of the cycle, part

of the water will be pumped from the Thermalito afterbay through Thermalito power plant back into the Forebay and the flow will be reversed in the Thermalito power canal back into the Thermalito diversion dam. The water will then be picked up by the pump turbines at the main plant and pumped back into the main Oroville reservoir.

THE RIVER AND THE WATERSHED

The Feather River is the most important tributary of the Sacramento River. It has a drainage area above Oroville of 3,611 sq miles. The seasonal runoff of the stream at that point varies from a mean of 4,244,000 acre-ft to a minimum of 1,200,000 acre-ft and a maximum of twice the mean. Flood flows have occurred up to a recorded maximum of 230,000 sec-ft. The smallest recorded flow was 300 sec-ft on November 9, 1931. This mean seasonal runoff represents about one-fifth of that of the entire Sacramento River drainage basin above the valley floor. A substantial part of the surplus waters of the Sacramento Basin lies in the Feather River area. Studies indicate that only about one-fifth of the mean seasonal runoff of the Feather River will be required to supply the ultimate water needs of its immediate service area when properly controlled and utilized. The remainder would be conserved to the extent practicable for exportation to areas of deficient water supply to the south.

Power and irrigation storage and diversion above the Oroville Dam site have a considerable influence on the runoff that reaches that point. The total upstream storage capacity in thirteen reservoirs amounts to 864,000 acre-ft. The Big Bend power plant is in the proposed Oroville Reservoir, and it is proposed to buy the plant and abandon it with the start of operation of the Oroville Reservoir.

In addition to the foregoing developments and diversions, the Oroville-Wyandotte Irrigation District is constructing a power and irrigation project on the South Fork of the Feather River above Oroville. The scheme will provide a storage capacity of 155,000 acre-ft in three reservoirs.

The Department of Water Resources of the state of California proposes to build five small to medium sized dams on tributaries of the Feather River that will be called the Upper Feather River Division of the Feather River Project. Their primary functions are conservation and recreation. Frenchman Dam is located on Little Last Chance Creek, a tributary of the middle fork of the Feather River. It is under construction (in 1961) and is an earthfill-type dam about 128 ft high that will impound a reservoir of about 50,000 acre-ft capacity. There are four other dams proposed for later construction in the Upper Feather River Division similar to Frenchman Dam.

PROJECT ACCOMPLISHMENTS

The Oroville Reservoir will provide a 750,000 acre-ft flood control reservation that will be sufficient to prevent another disaster such as occured in the Yuba City-Marysville area in December, 1955. Controlling the flow of the Feather River also will have important flood control effects throughout the lowlands along the Sacramento River and in the Sacramento-San Joaquin Delta. The Oroville and Thermalito Power Plants will generate 2,750,000,000 kw hr of electrical energy each year, much of it for peak demand. The annual yield

will be 1,135,000 acre-ft of water for use in the local service area as well as for export to the southern areas of deficiency.

THE DAM SITE

Rock types at the Oroville Dam site, that are of the Mesozoic Age or older, include amphibolite and small patches of meta-igneous granitic textured rocks. A few feet of residual soil covers these rocks between sparsely scattered rock outcrops on the abutments, while frequent floods tend to keep the rock near the river bottom washed and well exposed. There are sheared zones in each abutment that have been explored by means of diamond drill holes and also by exploration tunnels. The predominant foundation rock at the site is very hard, dense, green-gray to black, coarse to fine grained rock in which recrystallization of the minerals due to metamorphism is such that the original composition is obscure. The average specific gravity of the rock is about 2.96. The rock is generally massive although schistose in places. These rock types are cut by small veins of quartz and calcites. The rock in most cases is very hard, dense, and massive.

Quite an extensive program of diamond drilling exploration has been accomplished at the dam site and this has been supplemented by the driving of four exploration tunnels, two in each abutment. A special drilling program for determining the location of the underground power plant is under way (in May 1961), and it will be followed by a fifth exploration drift that will be driven into the proposed power plant location. Further detailed drilling will be accomplished from within this fifth exploration tunnel as well as a program of measurement of rock stress in place by means of flat jacks. An experimental grouting program has been carried out to get the basic information required for the design of the grout curtain and use as a basis for estimating grout take, and so forth, in the foundation. Exploration of the proposed borrow areas and an extensive laboratory testing program of the materials to be used in the dam are being carried out. As of May 1961, 220 holes have been drilled with a total footage of 24,852 ft of which 184 holes and 21,918 ft have been drilled at the main dam site. A total of 5,251 ft of exploration tunnel have been driven. Extensive auger and dragline pit explorations are being accomplished in the borrow areas. This has been supplemented by seismic surveys of the site. Cost of exploration to date represents about 0.3% of the estimated construction costs.

Seismicity studies of the proposed site were conducted to determine the history of earthquakes and their probable intensity at the site. During a period of 119 yr, twenty-two earthquakes have affected Oroville Dam site with an intensity of V or greater on the Rossi-Forel or Modified Mercalli scale. The December 27, 1869 earthquake, Rossi-Forel intensity VII, was the most severe earth vibration recorded for Oroville, and was probably the only shock that might have caused damage to modern structures at the dam site. Earth vibrations of intensity I through IV, apparently originating within a 20 mile radius of Oroville, may indicate small scale earthquake epicenters in the vicinity of the reported vibration. It is unlikely that any of this group of vibrations affected the Oroville Dam site. From this study it has been concluded that the site is in an area of relatively light seismic activity.

ALTERNATIVE DAM DESIGN STUDIES

In the studies leading to the selection of the final type of dam to be constructed at the site five different types of design were seriously considered.

The dam types were gravity, multiple arch, massive head buttress, arch-buttress, and the zone type fill dam that was finally selected. Preliminary designs and cost estimates were prepared for the five types of dams as a basis for the final decision. The gravity dam proved to be the most expensive, and the fill dam the least expensive, with the buttress type dams being somewhere between these two types.

The fill type design was selected not only because of economy but because the consulting board reached the unanimous conclusion that the site was not suitable for any type of concrete dam of the height required for this development.

FINAL DESIGN CHOSEN

The dam will be 735 ft high above stream bed and will contain approximately 80,000,000 cu yd of fill. (Fig. 2.) The main bulk of the dam will be the pervious sections that are made up of graded gravels up to 10 in. in size that have resulted from gold dredging activities in the vicinity of Oroville. There is a large area of these dredge tailings just southwest of the town of Oroville which is the proposed main borrow area for the dam. Just adjacent to the tailings area is a deposit of impervious material for use in the core of the dam. The coarser dredge tailing material is underlain by sands and silts that were separated from the tailing material in the gold dredging operation. It is proposed to use a combination of the underlying sands with a portion of the coarser dredge tailings to make up the transition material required in the section of the dam.

A novel feature of the design is the concrete core block section on which the impervious core rests. This limits the vertical height of the impervious core to 665 ft. Also provided is a concrete toe block at the downstream toe of the dam. The toe block will serve to contain the fill at the end of the second construction season when it is proposed to pass flood flows over the portion of the completed fill. Extending up the abutment from each side of the core block in the bottom of the core trench will be a grout cap of a size adequate to accommodate a grouting and drainage gallery from which the drilling and grouting for the grout curtain will be accomplished.

Diversion of the river will be provided for by means of two 38-ft ID tunnels through the left abutment that will, subsequently be plugged and used as tailrace tunnels for the underground power plant. This third construction year is the critical one for diversion, and it is planned to have the fill at El. 575 before the flood season of that year. With this storage available, the two diversion tunnels will be able to handle the standard project flood that is greater than twice the peak flow of the disasterous flood of December, 1955.

The section of the dam will be analyzed by both the wedge method and the slip circle method; because of its unprecedented height, conservative safety factors were selected. They are safety factor 2.0 for static conditions and 1.5 when the seismic force of 0.1 g is included in the analyses. Values of the angle of internal friction for the pervious and impervious materials of 40° and 30°, respectively, have been indicated by preliminary testing. An extensive program of exploration and testing of the materials in the borrow areas is under way at the present time. The length of haul from the borrow area to the dam is approximately 9 miles. Studies of the most economic method of hauling borrow materials from the pit to the dam site, have indicated that rail haul of material loaded in the pit by shovel is least expensive. This scheme makes

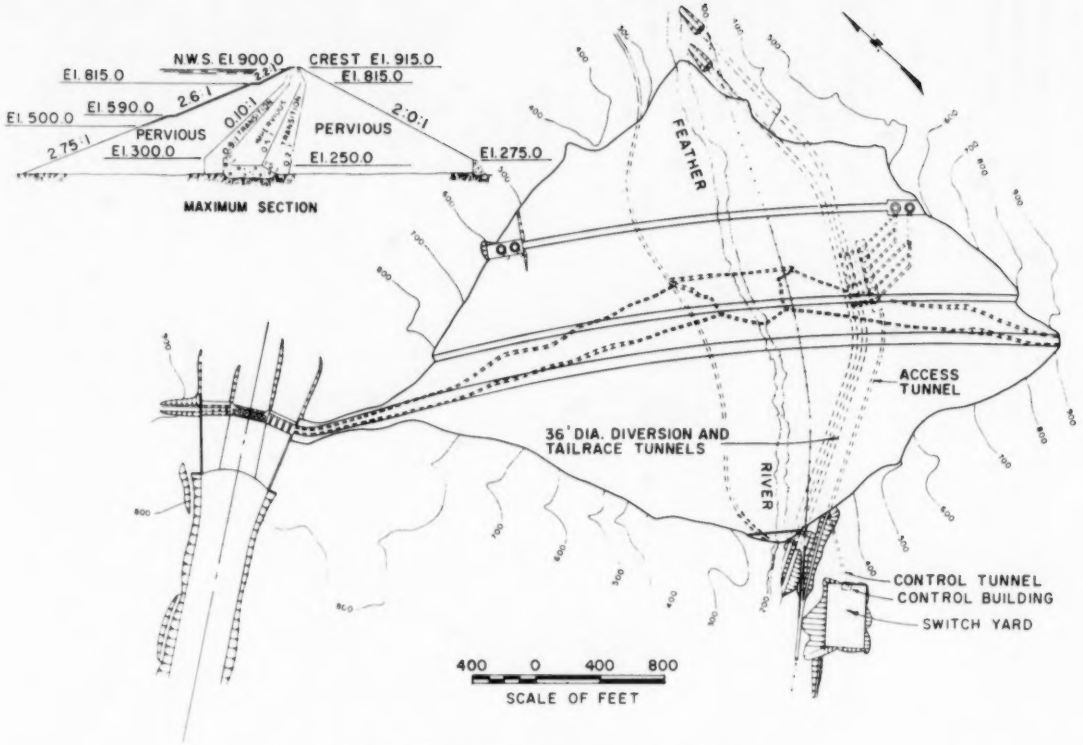


FIG. 2.—GENERAL PLAN-OROVILLE DAM

use of the former Western Pacific main line track that runs through the dam site and will become the property of the state of California when the new relocated line is put in service by the Western Pacific Railroad. The contemplated construction schedule requires a high rate of placement of fill materials that, of course, will require a very large spread of equipment by the construction contractor.

SPILLWAY

A chute type spillway is located in a saddle in the right abutment. Incorporated in the spillway, and centrally located, are seven 26.0 ft-by-26.67-ft top seal radial gates that will serve as flood control outlets. The regular overflow spillway will be located on either side of the flood control outlet works. It will have an ogee crest and will be provided with four 34-ft-by-95.0-ft radial gates, two on each side of the flood control outlet works. The combined capacity of the spillway and the flood control outlets in 509,000 sec-ft with water surface at El. 900 and 650,000 cfs with the reservoir at El. 909 which is the spillway flood pool. The outlet works gates would be electrically operated with manual controls. It is not planned to provide trashracks for these gates because they are of relatively large dimension and would pass ordinary floating debris without difficulty.

The spillway gates are of the "overhung" type (Fig. 3) similar to those in use at Horseshoe Dam in Arizona. With this type of design it is considered practicable to provide a gate width of 95 ft. The piers are located entirely downstream from the ogee crest. At one end each gate will seal against a pier or an abutment; at the other end it will seal against the other gate that will require a special sealing device, the details of which are to be worked out. The gate will be designed as a counterweighted gate with the operating force applied to the counterweight arm. Each gate will be provided with two counterweight arms, one at each pier, and will operate in a recess in the pier. To conserve lateral space and avoid making the pier wider than need be, the sides of the counterweight well are designed to be framed in steel. The gates will be designed for automatic operation and will be activated by a submersible counterweight hung in a well that will be open to the reservoir. When the water in the well tends to rise above mid-height of submersible counterweight, a force will be exerted opening the gate, and conversely, if the water should drop below mid-height of the submersible counterweight, the gates will close. Downstream of the gate structure proper there will be a paved concrete apron extending for a little over 400 ft that will terminate in a lip supported by a fairly massive concrete cutoff. From the end of the paved apron, an unpaved spillway channel cut to the rock will extend down to the river. An alternative design being considered is to provide a depressed section cut into the rock spillway chute sufficient to provide for the discharge from a project design flood of 150,000 sec-ft. Hydraulic model studies of the spillway were performed in the Hydraulic Laboratory of the University of California at Davis, Calif., from November, 1954 to March, 1955. However, changes have been made in the design since that time and it is planned to perform another complete model study of the present con-

cept of the spillway. At the downstream end of the spillway it will be discharging into the pool created by the diversion dam down stream of the main dam.

OROVILLE POWER PLANT

The Oroville Power Plant is being designed to accommodate a pump storage operation. Space will be provided in the power plant for the installation of six units with a total generating capacity of 600,000 kw, the initial installation to

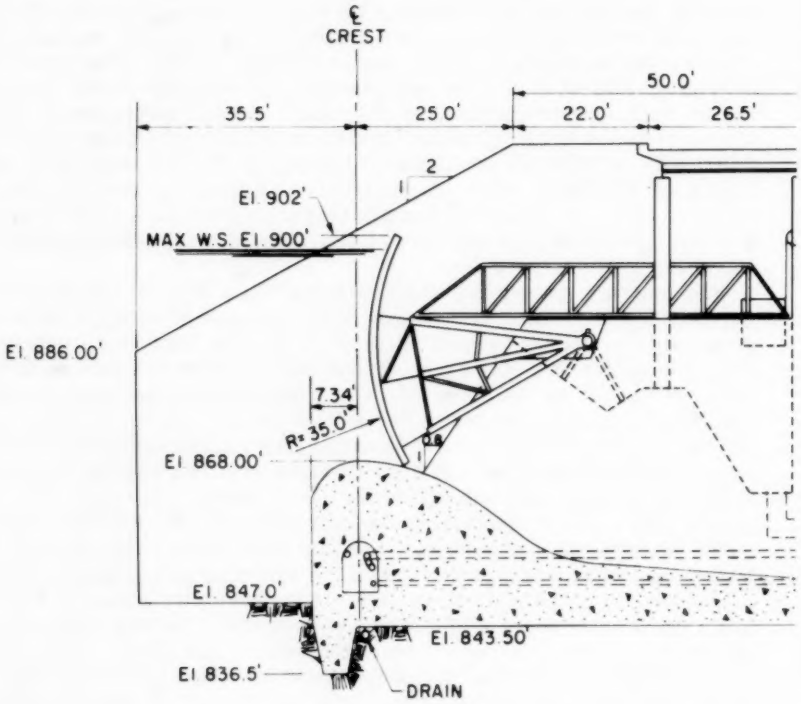
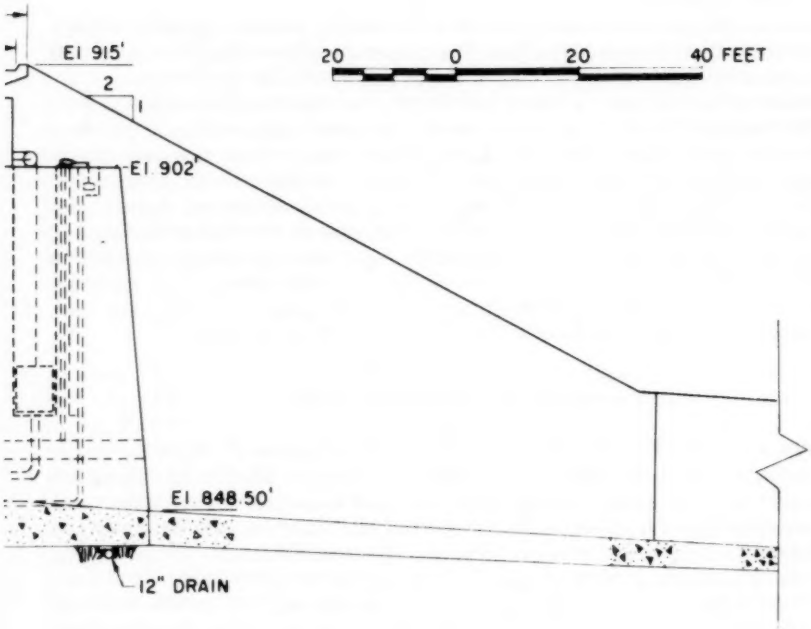


FIG. 3.—SECTION THROUGH

be operational in 1969. It is proposed to install three Francis turbine units and one reversible Francis type pump turbine unit having an aggregate generating capacity of 420,000 kw. When there is a market for the additional energy, it is proposed to install two more units of reversible pump turbines bringing the total generating capacity to 600,000 kw on a pump storage operation. The regular turbines will be rated as 110 megawatts. The pump turbines will be rated as 90 megawatts when generating and will be rated at 110 megawatts as motors when pumping.

Three different schemes for the power plant have been studied for this site. One scheme was to provide for the power plant in a large vault in the interior of the core block of the dam. This required a larger core block than presently shown to accommodate the vault required by the plant. Penstocks were in tunnels and also the tailrace was provided for in tunnels, some portions of which were cut-and-cover sections. A second scheme was to provide a surface type plant at the downstream toe of the dam. This scheme required rather long penstock tunnels that, in turn, required large surge tanks or pressure regulators, both of which increased the cost of this scheme considerably. The third



SPILLWAY GATE

scheme was to provide an underground power plant excavated in the left abutment of the dam with the penstocks constructed in tunnels and the tailrace also taken care of by tunnels that would be used as diversion tunnels in the early part of the construction. The underground plant was selected on the basis of least cost and best operating characteristics.

The main machine hall will be about 513 ft long with a 72-ft clear span and a height of approximately 125 ft from the crown to the bottom of the draft tube excavation. Two 22-ft diameter tunnel penstocks will deliver the water to the

turbines. The reservoir end of the penstock tunnels will be controlled by bouyant bell type gates that will be operated by compressed air from the power plant. These will serve as emergency gates. The draft tubes of the turbines will discharge into the two 38-ft ID diversion tunnels. The higher of the two tunnels will provide a free water surface within a reasonable distance of the unit. Surge studies are presently under way for the entire system of the penstock tunnels and tailrace tunnels, and a hydraulic model of the system is planned.

Transformers (13.8/230 KV) will be located between the turbine units and the leads will run through a compartment in an access tunnel to the switchyard that will be located at the downstream toe of the dam.

Two additional intake structures and two stub penstock tunnels extending down to sound rock will be provided for future expansion of the plant when that is considered advisable.

Bypasses will be provided through the tunnel plugs; one a temporary bypass used during construction and the other a permanent bypass to satisfy irrigation needs in the event that the power plant would be completely shut down.

It is planned to provide a control building at the downstream toe of the dam located at the switchyard. From this location, the main units will be controlled as well as the units in the second power plant downstream from the main plant. Other plant layouts are under study, including one with the units arranged in a circle in plan. As of April, 1961 the final decision on layout had not been made.

The pump turbines would be operated as pumps in the winter and spring months of dry years to return to the Oroville reservoir the power releases in excess of downstream water requirements. The Oroville power plant turbines would be designed to assure full dependable peaking capacity at that plant with the Oroville reservoir drawdown to 1,200,000 acre-ft of storage.

THERMALITO DIVERSION DAM

The Thermalito diversion dam will be constructed about 4½ river miles downstream from the main dam. It will back up a pool to El. 225 and establish the tailwater on the Oroville Power Plant. Its pool will also be the plunge pool for the spillway for the Oroville Dam. It will be a concrete gravity structure about 150 ft in height above stream bed and will serve to divert the water into the Thermalito Canal. It will be 1,280 ft long containing 127,000 cu yd of concrete. Its overflow spillway will be equipped with seven 21-ft-by-90-ft radial gates. The head gates for the Thermalito Canal will be three 33-ft-by-33-ft topseal radial gates. It is necessary to release about 400 cfs of water through this dam in order to keep the river downstream of the dam alive. This water is passed through a small power plant that will be built inside the gravity dam itself. This plant will generate station power that will be used at the two larger power plants and for other project purposes.

THERMALITO CANAL AND FOREBAY

The Thermalito power canal will have a capacity of 18,000 cfs, and it conveys the water westward from the diversion dam to the Thermaltio forebay. It has a bottom width of 48 ft and a maximum depth of 23 ft. The canal is de-

signed to carry flow in the reverse direction when the pump turbines at the Thermalito plant are operating on the pumping cycle.

The Thermalito Forebay Dam is a low earthfill structure with a maximum height of about 65 ft that backs up a reservoir of about 14,000 acre-ft.

THERMALITO POWER PLANT

The Thermalito power plant will have a capacity of 105,000 kw and will be equipped to operate for pump storage with one regular type turbine and two pump turbines. It is planned to double this capacity when justified.

THERMALITO AFTERBAY

The tailrace channel of the Thermalito power plant discharges into the Thermaltio afterbay which is contained by a long low earth dam. The afterbay has a capacity of 44,500 acre-ft and provides the necessary reregulation for the discharge of a reasonably uniform flow back into the natural river channel at the outlet structure. Irrigation water is supplied at two different points in the afterbay, on the Western Canal headworks and the other the Sutter-Butte Canal headworks. During the pumping cycle water from the afterbay is pumped back through the Thermalito power plant and the Thermalito power canal to the pool behind the diversion dam where it is picked up by the pumps in the main power plant and pumped back into Oroville reservoir.

UTILITY RELOCATIONS AND FISH FACILITIES

Some costly utility relocations are involved in the construction of the project. In addition, 20.5 miles of state highway (alternate U. S. 40), 27.1 miles of the Western Pacific Railroad, and six miles of the Feather River Railway must be relocated. Also, the Pacific Gas and Electric's Big Bend power plant with 70,000 kw capacity will be completely inundated. There are county roads, fire protection roads, power transmission, telephone and telegraph lines, and the Palermo Canal of the Oroville-Wyandotte Irrigation District that will have to be relocated or modified. A fish hatchery and artificial spawning beds will have to be constructed downstream of the Thermaltio Diversion Dam just across the river from Oroville to provide for the salmon and steelhead run of the Feather River.

The estimated cost of the project is $425,000,000 of which $250,000,000 is for the main dam, power plant, and spillway, $55,000,000 for the downstream facilities and $120,000,000 for the cost of rights of way and relocations. The acquisition of lands and improvements and relocation of utilities is well under way. As of April, 1961 there has been about $60,000,000 spent or earmarked for these features of the project.

ACKNOWLEDGMENTS

Harvey O. Banks, F. ASCE, is Director of California's Department of Water Resources which is responsible for the design and construction of Oroville

Dam. H. G. Dewey, F. ASCE, is Chief Engineer, Division of Design and Construction and D. P. Thayer, F. ASCE, is Assistant Division Engineer, in charge of design. The department's consulting board for the dam is as follows: Raymond A. Hill, F. ASCE, John J. Hammond, F. ASCE, Byram W. Steele, M. ASCE, B. E. Torpen, F. ASCE, and Roger Rhoades, F. ASCE.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

GLEN CANYON POWERPLANT[a]

By Samuel Judd,[1] F., ASCE

SYNOPSIS

The principal features of the Glen Canyon Powerplant, which is a major structure of the Glen Canyon Unit of the Bureau's Colorado River Storage Project in northern Arizona is described. The paper discusses location studies, reconnaissance for the powerplant site, specifications, preconstruction considerations, structural design, and architectural design of the powerplant.

INTRODUCTION

The Glen Canyon Powerplant will be one of the principal features of the Glen Canyon Unit of the Colorado River Storage Project. This project was authorized by Congress in the spring of 1956 to develop the land and water resources of the 110,000-sq-mile area called the Upper Colorado River Basin. The Glen Canyon Unit is located on the Colorado River in northcentral Arizona, 15 miles upstream from Lees Ferry and 13 river miles downstream from the Arizona-Utah state line, Fig. 1.

In addition to the Glen Canyon Dam and Powerplant, the unit includes access roads, a switchyard, the Glen Canyon Bridge, and the new permanent community of Page, Ariz. This town is named after the late John C. Page, Hon. M., ASCE, and Commissioner of Reclamation from 1937 to 1943. The community

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

[a] Presented at the June, 1960 ASCE Convention at Reno, Nev.

[1] Chf., Structural and Archit. Branch, Division of Design, Bur. of Reclamation, Denver, Colo.

will house both the contractor's forces and the Bureau's personnel during construction, and, subsequently, the operation and maintenance forces. The total cost of the Glen Canyon Unit, without transmission facilities, is estimated at around $326,000,000.

Construction of the dam and powerplant at Glen Canyon began 1 yr after the project was authorized by Congress. A low bid of $107,955,522 was submitted and the contract was awarded on April 29, 1957. This represents one of the

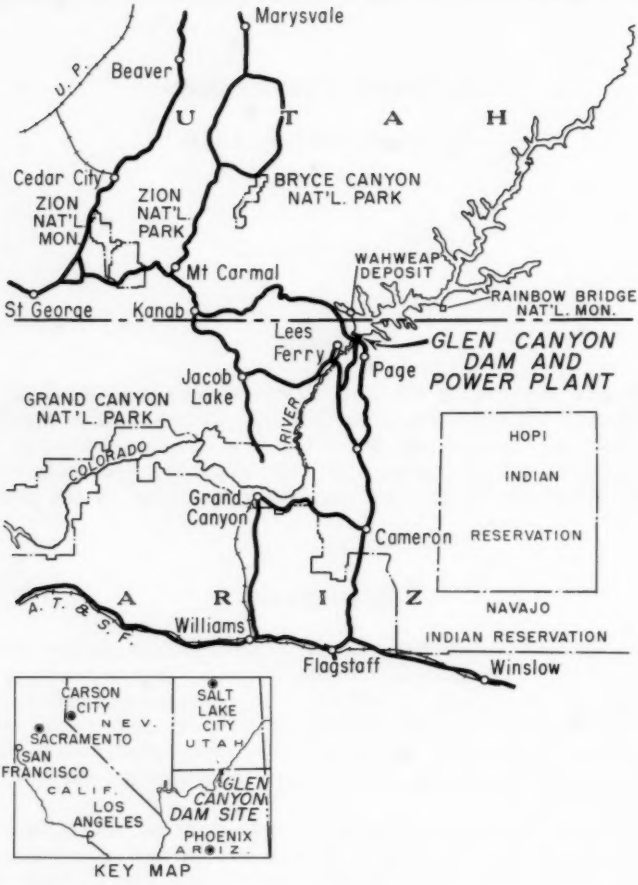


FIG. 1.—LOCATION MAP

largest single contracts on record for a dam and powerplant, and the largest ever awarded by the Bureau of Reclamation.

Glen Canyon Dam will rise about 700 ft above the foundation and create a reservoir of 28,000,000-acre-ft capacity. The powerplant will be constructed 400 ft downstream from the axis of the dam, Fig. 2. It will be a reinforced

concrete structure about 650 ft long, 113 ft wide, and 150 ft high above the mass concrete required for its foundation. The ultimate installation will comprise eight 112,500-kw generating units, each driven by a 155,500-hp, vertical shaft hydraulic turbine. Water will be delivered to each turbine through a 14-ft-diameter steel penstock. The total generating capacity will be 900,000 kw, or about 81% of that authorized for the entire project.

LOCATION STUDIES

A record of the foundation investigations and testing has been presented by Louis G. Puls,[2] F. ASCE. The area is essentially a country of broad, cliff-edged mesas cut by narrow, steep-walled canyons. The site selected was a narrow section of Glen Canyon with nearly vertical walls rising about 700 ft above the Colorado River.

The climate in the vicinity is dry. Temperatures range from 114° F in summer to 2° F in December and January. Average snowfall is 4 in. recorded at Lees Ferry. The annual precipitation averages 6 in.

General.—Several difficult problems are presented in fitting a long multi-unit powerhouse into a narrow canyon in conjunction with a concrete arch dam. To position the powerplant parallel to the canyon walls requires the use of large tunnels in the canyon walls leading to smaller tunnels to feed water to each unit. This method of bringing water to each of the hydroelectric units has to be compared with the design using penstocks through the dam to a building placed across the canyon. This latter is a careful balance of the shortest penstocks possible for economy with the design requirement that the excavation for the powerplant end bays will not remove rock that is needed for support of the arch dam at its abutments, Fig. 3.

At Glen Canyon, the use of expensive intake towers in the reservoir and tunnels in the rock was discarded in favor of penstocks through individual blocks in the dam with intakes on the face of the dam. This decision to place the powerhouse across the canyon, however, caused many difficulties in the building layout and influenced to a great extent the number of hydroelectric power units.

PHASES

The functional design leading to finished general arrangement drawings of the Glen Canyon Powerplant progressed through three distinct periods, namely: reconnaissance, specifications, and preconstruction. Comparative estimates were prepared where necessary to select the best of several schemes in each phase.

Phase I—Reconnaissance.—A favorable site for a major dam and powerplant had been selected in the field in Mile 15, or 15.3 miles upstream from Lees Ferry on the Colorado River.

In 1949 and 1950, most of the design data were far from being firm. The height of the dam had not been determined and consequently the reservoir storage capacity and the power head had to be assumed in order to start on layouts for "reconnaissance-type" estimates. A preliminary draft of an interim report in 1949 had recommended a powerplant of 800,000-kw capacity.

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[2] "Design of Glen Canyon Dam," by Louis G. Puls, presented at the February, 1959 ASCE Convention at Los Angeles, Calif.

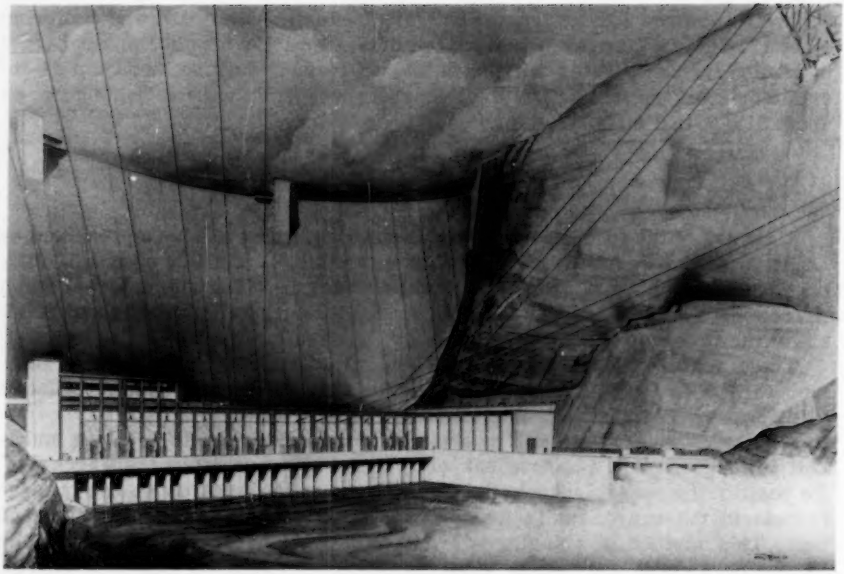


FIG. 2.—GLEN CANYON DAM AND POWER PLANT

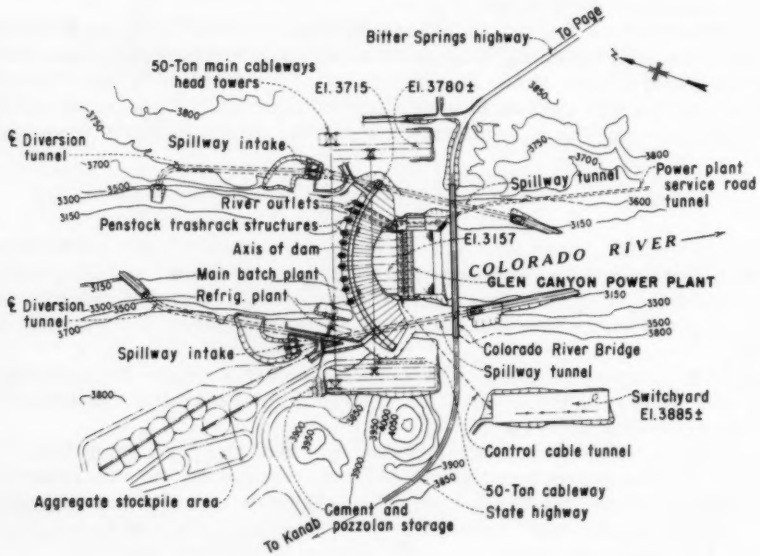


FIG. 3.—AREA PLAN

One of the problems in establishing the number of units in the powerplant was their effect on the dam design. The number of penstocks would affect the position of the intakes, the outlet pipes, and the jointing of the dam. Three schemes were made of 7-unit surface plants with 138,600-kw units for a 970,200-kw total capacity. One surface was for a surface plant with six 138,600-kw units for a total capacity of 831,600 kw.

A layout was made of an underground plant downstream from the dam's left abutment. It consisted of an excavated hall containing 6 units of the same capacity as the 6-unit surface plant, 2 station-service units, with a service bay and a control bay at opposite ends of the hall. The transformers were to be located in partitioned vaults along the generator floor. Rough estimates of comparative cost of the 6-unit surface and underground schemes were made, and on the basis of these estimates, no further consideration was given to the underground scheme.

Phase II—Specifications.—As the specification phase of the work was entered into, schemes for 6-, 7-, 8-, and 10-unit plants were considered. The building for the 10-unit plant was approximately 745 ft long. An excessive amount of canyon wall rock excavation would have resulted from a cross-canyon position but to have turned it parallel to the canyon wall would have required expensive water tunnels in the canyon walls. Although the 10-unit plant required slightly smaller bays, the larger number of units resulted in a plant longer than the 7- and 8-unit plants. The cost of the penstocks for the 10-unit scheme was almost twice that for the 7-unit scheme because of their greater length and number, although they were smaller in diameter.

Therefore, the use of more than 8 units appeared undesirable from the economical and physical standpoints.

After careful consideration of the aforementioned schemes, it was decided to study a plant with eight 112,500-kw units with a total capacity of 900,000 kw, and a design head of 510 ft. A study of the effect on the building size for machine speeds of 150 rpm and 163.6 rpm showed a somewhat smaller size unit for the faster speed. However, it was decided to use the slower speed of 150 rpm to avoid any possible generator design difficulties. This 150-rpm speed, design head of 510 ft and rated head of 450 ft, the number and size of units at 8, with 112,500-kw capacity each, became the statistics used throughout all later schemes and the specifications, Fig. 4 shows a transverse section thru unit No. 1.

The experience gained from previous schemes for long plants showed that long penstocks were necessary for a location which avoided the dam thrust lines. Also, such a location at Glen Canyon resulted in considerable depths of mass concrete fill under the powerplant. Therefore, a scheme was studied to shorten the building by placing the control room and appurtenant rooms in an upstream lean-to, instead of in an end bay. Then a layout was made with the control room on the generator floor level of the service bay, one story below the entry or erection floor instead of the previous upstream location.

A cost study of the mass concrete foundations comparing cellular wall supports with mass concrete fill revealed that the mass concrete fill was less expensive and, therefore, it was retained in the design.

The general arrangement plan as incorporated in the construction specifications shows the transformers on a downstream deck, the control bay on the right side, the expansion joints between each unit, a powerplant length of 665 ft, and the machine shop upstream of the service bay. It was also decided to

locate the governor gallery on the downstream side of the plant. However, as the preparation of these drawings was under way, it was possible to make certain improvements. By checking the building for clearance with the thrust lines of a new dam design, it was possible to move the building closer to the dam axis, from 524 ft that had been previously established to the indicated 470 ft.

Phase III—Preconstruction.—The plant, as shown on the specification drawings, met the requirement of positioning all powerplant excavation outside the thrust lines in the abutments supporting the concrete arch dam design shown

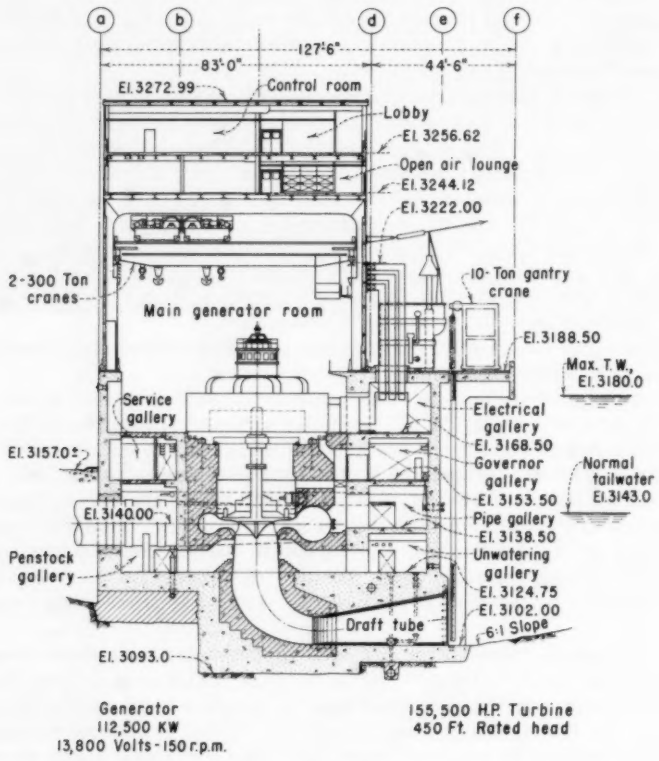


FIG. 4.—TRANSVERSE SECTION THRU UNIT NO. 1

in the specifications. However, the final reports of additional tests taken subsequent to the preparation and issuance of the specifications required thickening the dam at the abutments to reduce the bearing pressures. This revised dam design changed the arch thrust lines in such a way as to require locating the powerplant downstream an additional 20 ft, or 490 ft total from the dam axis. Unless the designers were able to arrive at a compensating design, this 20-ft shift would have lengthened the 8 penstocks and 4 outlet pipes and would have increased the powerplant cost. Therefore, it was decided that studies

should be made not only to avoid shifting the powerhouse the 20 ft downstream, but that an attempt be made to reduce the original 470 ft shown in the specifications and thus make further savings. It was decided to modify the positions of the control and service bays and the machine shop, in relationship to the generating units, that resulted in an L-shaped building. Various modified L-shaped schemes were drawn.

Moving the machine shop from the upstream specifications location to downstream of the service bay was studied. This reversal of the "L" allowed the service bay and eight units to move closer to the dam and shortened the penstocks, but in so doing caused the control bay on the right end to encroach on the dam thrust lines.

The control bay, as such, was eliminated except for a narrow tower located at the downstream corner of the end of Unit 1. This tower contained a stairway, a passenger elevator, and a cable shaft. A 2-story structural steel structure called the control area was placed above the supersturcture of Units 1 and 2. This shortened the length of the upstream wall of the building and permitted this wall to be located 400 ft from the dam axis. While the elevator tower added 24 ft and the widened machine shop added 25 ft to the overall length of the building, these extensions were downstream of the upstream powerplant wall and outside of the abutment thrust lines.

FINAL DAM AND POWERPLANT DESIGN

A review of the layouts, made in conjunction with previous dam designs, showed that with minor modifications the result of these last powerplant studies could be utilized for the final design of the powerplant structure in conjunction with the final dam design.

New general arrangement drawings were prepared superseding all or part of the specifications drawings. The final design, Figs. 5 and 6, for construction shows:

1. The upstream wall of the units and service bay located 400 ft from the axis, that is 70 ft less than the specifications design.

2. The final length to be 598 ft 9 in. for the upstream or "a"-line length and 648-ft 9-in. overall length from the elevator tower to the machine shop.

3. The machine shop located downstream instead of upstream of the service bay with a passage on the tailrace side of the machine shop for access to the service bay.

4. Elimination of the control bay formerly adjacent to Unit 1. The elevators, stair tower, and cable shaft were placed in Unit 1.

5. The control room, cable spreading room, and offices formerly in the control bay located above the typical superstructure of Units 1 and 2.

As a result of these studies in the Phase III schemes, this location producing shorter penstocks, together with the layout improvements, and using the bid unit costs, saved an estimated $1,000,000 over the specifications design.

STRUCTURAL DESIGN

Stability.—The unit bays are separated from each other by a contraction-expansion joint system. This insures that they act as separate monoliths. The

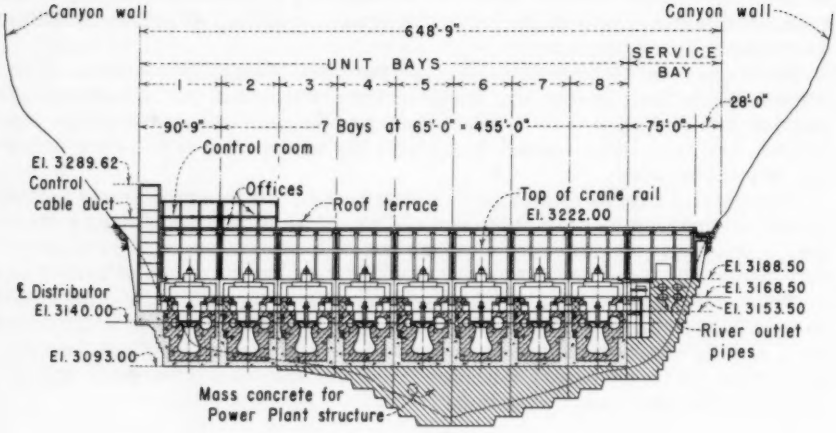


FIG. 5.—LONGITUDINAL SECTION THRU CENTERLINE UNITS

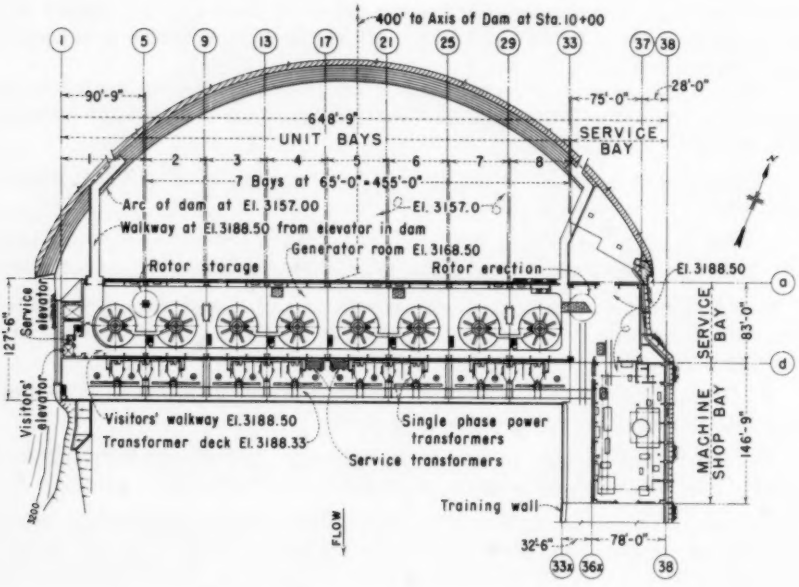


FIG. 6.—GENERAL PLAN-EL. 3188.50

joints are located 65 ft on centers as is customary with units of this size. In addition, the mass concrete required beneath the unit bays was necessarily jointed at the same intervals. While consideration was given to combining two unit bays into one monolith, the mass concrete and magnitude of the units precluded this arrangement.

The unit bay monoliths comprising the mass concrete in the powerhouse structure were analyzed as outlined in another publication.[3] The allowable factors of safety are divided into two parts for the various loading conditions consisting of the skeleton powerhouse structure, the completed structure, maximum tailwater, minimum tailwater, saturated backfill, and earthquake. These are shown in Table 1.

TABLE 1.—ALLOWABLE FACTORS OF SAFETY

Factors of Safety	During Construction		Structure Completed and Equipment Operating	
	Loading			
	Normal	Extreme	Normal	Extreme
(1)	(2)	(3)	(4)	(5)
Overturning	1.1	1.1	1.2	1.1
Shear-friction	2.5	1.1	3.5	2.0
Flotation	1.1	1.1	1.2	1.1

The factors of safety were defined by the following

$$\text{Overturning} = \frac{\Sigma RM \text{ (righting moment)}}{\Sigma OM \text{ (overturning moment)}} \quad \ldots\ldots\ldots (1)$$

$$\text{Shear-friction} = \frac{(\Sigma W+U)f + cA}{H} \quad \ldots\ldots\ldots\ldots (2)$$

and

$$\text{Flotation} = \frac{\Sigma W \text{ (vertical forces)}}{\Sigma U \text{ (uplift forces)}} \quad \ldots\ldots\ldots\ldots (3)$$

in which ΣW is the Summation of vertical forces; c denotes cohesion, in kips per sq ft; A represents area, in square feet of base in compression; U denotes uplift forces; H represents horizontal forces; and f is the coefficient of friction.

The properties of the foundation rock were determined by laboratory testing as follows:

a. Coefficient of friction, "f", of concrete to rock = 0.6.

b. Unit shearing strength (cohesion) of rock in powerplant area = 300 psi. Concrete on rock was taken = 100 psi.

c. Bond strength in tension between concrete and rock equal to 70 psi. In the analysis, this property was not utilized because of its probable lack of uniformity.

d. The allowable bearing values, concrete on rock, were established at 86 kips per sq ft without earthquake and 100 kips per sq ft with earthquake.

[3] "Design Criteria for Concrete Gravity and Arch Dam," Bur. of Reclamation Engineering Monograph No. 19, 1960, Dept. of the Interior.

The analysis, as defined by the aforementioned criteria, resulted in overturning, shear-friction, and flotation safety factors above the allowable values listed in Table 1. The minimum overturning factor was 1.1, the minimum shear-friction value was 2.7, and the minimum flotation value was 1.2.

Substructure.—The substructure of the powerplant was designed for the foundation pressures and uplift values determined from the stability analysis. Inasmuch as the concrete required around the draft tube and turbine pit is usually proportioned on the basis of shearing values, the amount of reinforcement is often determined from shrinkage and volume change computations. To minimize the amount of steel required, two positive control measures were adopted. Placing temperatures were limited to 50° F and construction joints were located to minimize the length of placement. It was found that a carefully planned construction point sequence contributes more to the elimination of shrinkage cracking in the substructure than extensive reinforcement.

Intermediate Structure.—The intermediate structure of the powerplant extends from El. 3124.75 to El. 3188.50 (Figs. 7, 8, and 9). Its analysis was divided into two parts. Part I consists of the downstream gallery system extending between unit bay crosswalls. Part II consists of the upstream gallery system extending between unit bay crosswalls. The framing for the downstream gallery system is designed to resist maximum tailwater, dead and live loads from the mechanical and electrical equipment, temperature change, and earthquake. The upstream gallery system is similar except that the backfill to El. 3157 replaces the afterbay water load. A three-dimensional analysis of these framing systems was made using a modified trial load analysis. The applied loads were divided between a grid of the principal horizontal and vertical elements of the structure that maintained continuity of the elements. Major horizontal elements consisted of the floors and major vertical elements consisted of piers and columns. The effects of shearing deformations and yielding at supports as well as variable sections were included. Instead of utilizing the trial process often used in the trial load analysis, the solution of load distribution was determined by use of a system of simultaneous equations set up for solution by the electronic data processing machine, and IBM 650.

The transformer deck design presented a difficult problem in that clearance requirements required this area to be framed as a flat slab. Due to the variable support conditions, the large openings required for the isolated phase bus, and the heavy concentrated wheel loads of the transformers, an electronic data processing machine analysis was again used to save time and permit more accurate results. The analysis was performed by a new approach using a load distribution method similar to the trial load method of analysis. The applied load on a transformer deck was divided between a grid of sample structural elements replacing the slab and adjusted in a manner to maintain continuity of deflection in one direction. Instead of accomplishing this by "trial," properties of the unit-distributed loads were first computed, and then a system of linear simultaneous equations was set up for solution by the high speed computer. A portion of the basic data computations was also programmed for computer solution. These computer programs are general, that is, they are applicable for solution of future structural elements of the same type.

Superstructure.—The superstructure of the powerplant is a structural steel frame with concrete walls and lightweight precast concrete roof slabs. The principal structural members of the superstructure are welded built-up rigid frame steel bents, supporting welded steel crane runway girders. The crane

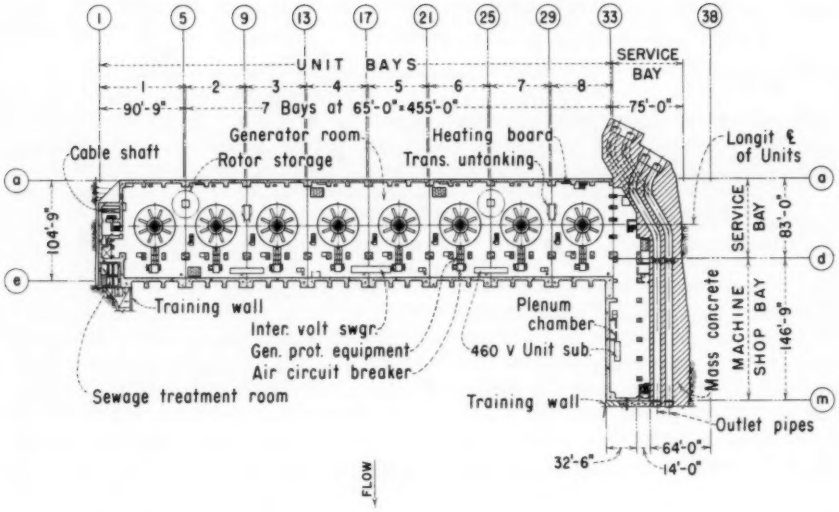


FIG. 7.—GENERAL PLAN-EL. 3168.50

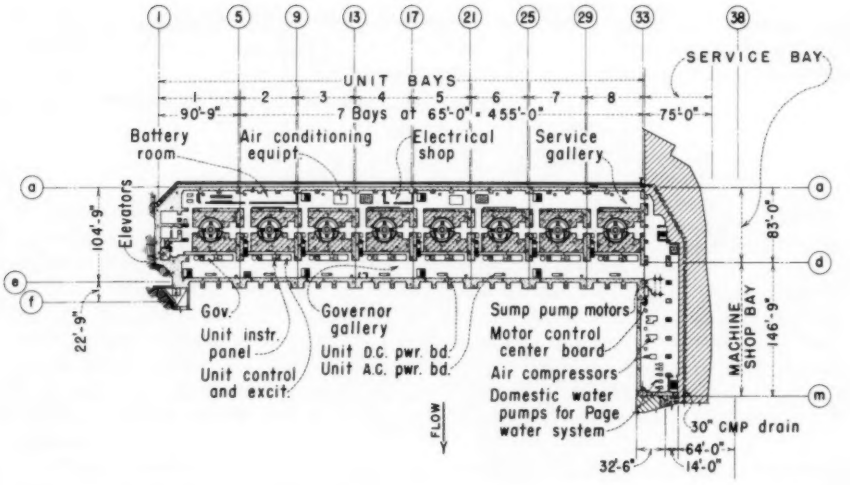


FIG. 8.—GENERAL PLAN-EL. 3153.50

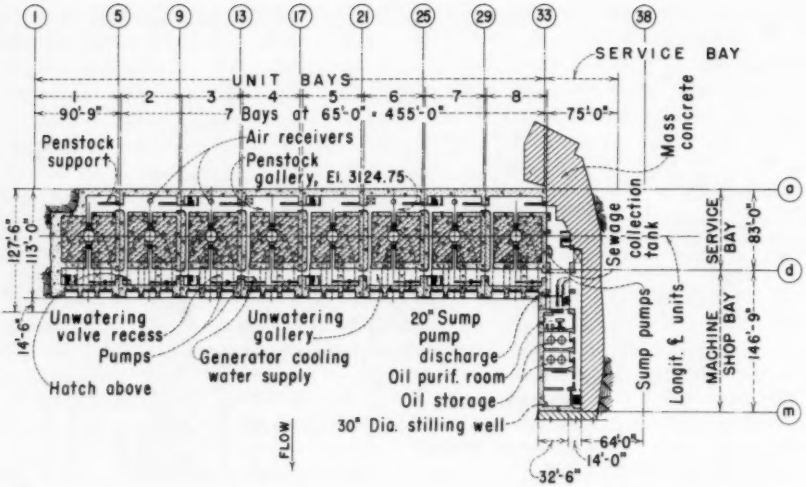


FIG. 9.—GENERAL PLAN-EL. 3124.75

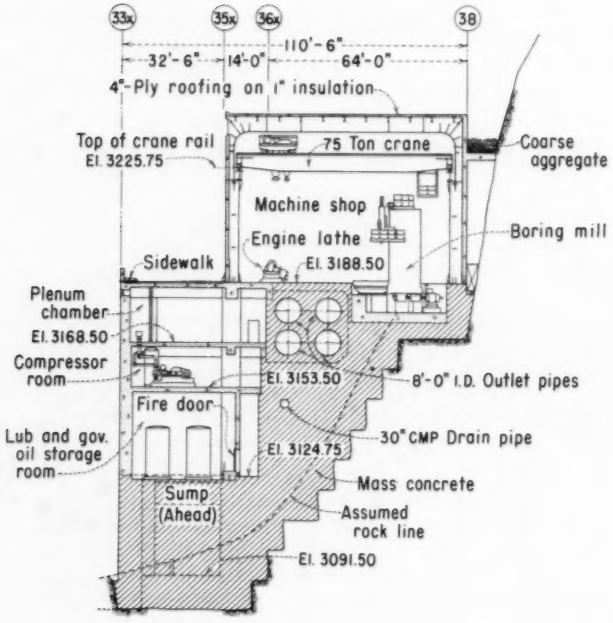


FIG. 10.—TRANSVERSE SECTION THRU MACHINE SHOP BAY

runway girders carrying two 300-ton bridge cranes are designed as continuous over the supports. Rocker plates welded directly to the bottom flanges of the crane girders transmit the crane load reactions onto cap plates at each column. The following are special points of interest about the powerplant superstructure:

1. The outside steel column flanges are exposed, the concrete walls being on the inside of the column flange.
2. The contractor was given the option of using precast concrete walls or cast-in-place concrete, and elected to use the latter.
3. The parts of the structure adjacent to the canyon wall have cast-in-place concrete roof slabs as a protection against falling rocks.
4. The control area is a two-story steel frame structure over the powerplant roof above two of the generator bays.

The roof of the machine shop and service bay adjacent to the left abutment is designed for cast-in-place concrete to better resist impact from falling debris (Fig. 10). In addition, a debris trap has been located along the left abutment between the rock and the service bay and machine shop structures to contain the majority of falling or spalling rock. No provision was made along the right abutment for this contingency because the roof of the unit bay is at a greater distance from the abutment. The remainder of the roof over the bays is composed of precast concrete slabs covered with built-up roofing. A summary of the loads and unit stresses is given in Fig. 11

Second Stage Concrete.—The second stage concrete utilized for encasing the turbine scroll case and supporting the generator will be installed under the completion contract for the powerplant. The two essential parts of the design are confined to the turbine encasement and the generator supports. The turbine scroll case will be embedded under normal operating pressures, and the concrete encasing the turbine will be reinforced for the water-hammer pressures.

The generator foundation design will transmit the vertical loads to the supporting first stage concrete below. Lateral loads from braking, short-circuit torque, and synchronizing torque will be resisted by shear and bearing in the second stage foundations block.

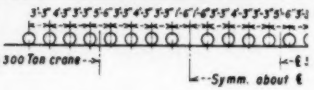
ARCHITECTURAL DESIGN

The exterior architectural treatment of the powerplant is composed of concrete surfaces, exposed flanges of structural steel columns, and metal window walls. The concrete surfaces of the cable and elevator tower and the downstream wall of the machine shop are to be formed with Tongue and Groove sheathing. Concrete wall surfaces of the unit bays, the west wall of the machine shop, and the upstream wall of the service bay will be formed with plywood form material. The plywood panels will be approximately 10 ft long by 4 ft high and placed to provide a stacked pattern in the finished concrete surfaces. Plywood panels will be butted together and thus provide a subdued joint pattern.

Fixed metal windows, extending from column to column, are placed directly below the roof eaves of the unit bay, machine shop, and service bay walls. The horizontal placement of these windows, the plywood formed concrete surfaces between the exposed steel column flanges, and the Tongue and Groove formed

LOADS					
ELEV.	LOCATION	LIVE LOAD LB./SQ. FT.	SPECIAL LOADS	FLOOR FINISH OR SURFACING	WEIGHT FL. FIN. LB/SQ. FT.
	CABLE AND ELEVATOR TOWER				
3289.62	Roof	50		4-Ply roofing	
3279.12	Elevator equipt. floor	250		6" Conc. surfacing	75
3267.87	Storage	150			
3262.87	Toilet and janitor room	100			
3256.62	Elevator lobby	200			
3256.62	File storage	150			
3250.37	Toilet and janitor room	100			
3244.12	Elevator lobby	200			
3244.12	Cable shaft	100	Cables		
3239.12	Toilet and janitor room	100			
3233.50	Elevator lobby	100			
3227.88	Office and storage	100			
3222.25	Elevator lobby	100			
3216.63	Telephone room and storage	150			
3211.00	Elevator lobby	100			
3205.38	Office, storage and cable shaft	100			
3199.75	Elevator lobby	100			
3194.13	Toilet and janitor room	100			
	CONTROL AREA				
3256.64	Lobby	200			
3256.64	Control room	200			
3256.64	Offices	100			
3256.62	Walkway	200			
3244.14	Lobby	200			
3244.14	Dispatching room	200			
3244.14	Offices	100			
3244.12	Walkway	200			
3244.12	Observation deck	200			
	UNIT BAYS				
3243.88	Roof for Units 4 thru 8 (14)-(18)	50		4-Ply roofing	
3188.50	Transfer deck (8)-(7)	500	B,C,F	6" Cover slab	75
3188.50	Transformer deck (4)-(8)	500	C,F	6" Cover slab	75
3188.50	Balcony (18)-(14)	1,000	E,F,G,J	4" Conc. surfacing	50
3188.50	Balcony - all others	200		4" Conc. surfacing	50
3183.50	Toilet and janitor room (1)-(2)	100			
3168.50	Generator floor (2)-(4)	1,000	D,E,G	4" Conc. surfacing	50
3168.50	Electrical gallery (4)-(8)	500*	D,H	4" Conc. surfacing	50
3168.50	Toilet and janitor room (1)-(2)	100			
3153.50	Governor gallery (4)-(8)	500	D	4" Conc. surfacing	50
3153.50	Service gallery (8)-(5)	300		Monolithic	
3153.50	Sewage collection and storage (1)-(2)	100			
3138.50	Pipe gallery (2)-(6)	300		Monolithic	
3124.75	Unwatering gallery (2)-(6)	200		Monolithic	
3124.75	Penstock gallery (8)-(5)	Unlimited		Monolithic	
	Stairs - below El. 3153.50	100		Monolithic	
	Stairs - above El. 3153.50	100		2" Bonded conc.	25

A-POWER HOUSE CRANE-TWO 300 T



104 Kips per wheel max. + 10.4 Kips per wheel
47 Kips per wheel min. + 4.7 Kips per wheel
Longit. and lateral thrust 10.4 Kips per wh
side and 4.7 Kips per wheel opposite side.

Wt. of crane = 505 Kips

LOADING CONDITIONS FOR DESIGN

1. Dead loads.
2. Live loads on various floors and roof.
3. Wind loads.
4. Crane loads and thrusts.
5. Earthquake forces.
6. Temperature variations.
7. Backfill loads.
8. Maximum tailwater.
9. Normal tailwater.
10. Minimum tailwater.

ASSUMPTIONS FOR STABILITY ANALYSIS

Case I. All first stage concrete and superstructure completed; draft tube unwatered; normal tailwater; backfill upstream side to El. 3125.00 and assumed dry above El. 3020.00 with and without full uplift over entire base. Required factor of safety against flotation = 1.10.

Case II, IIA, IIB, IIC, III, IIIA, IX, IXA, & X For loads see Dwg. 557-D-603 & 604

UNIT DATA

Generator rating	112,500 kw
Volts	13,800
Speed	150 rpm
Turbine rating	155,500 hp
Rated head	450 ft
Turbine discharge	3,580 cfs

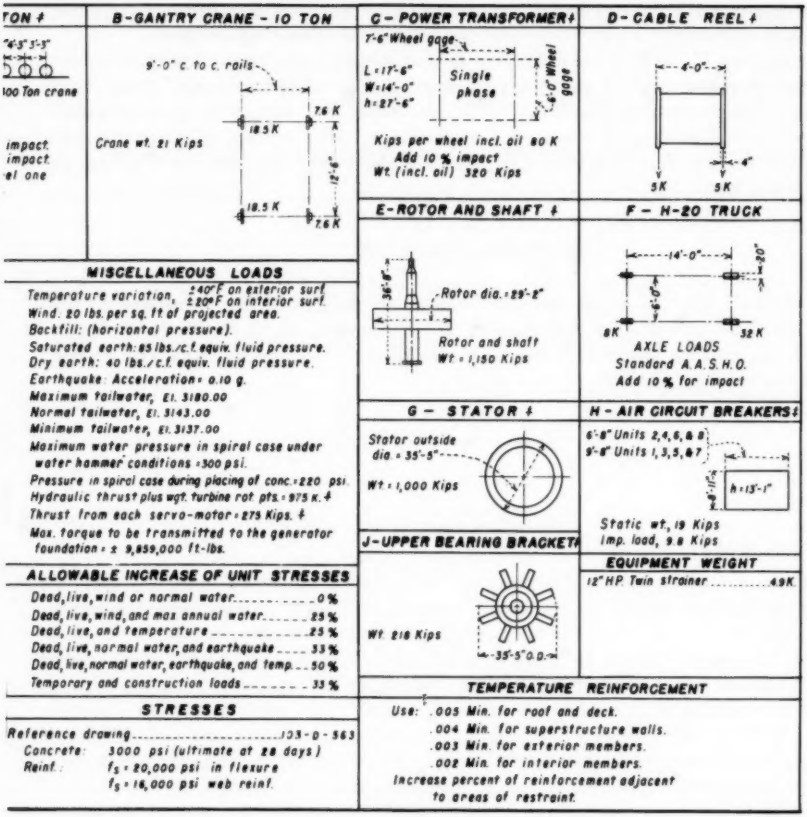
FIG. 11.—SUMMARY OF

concrete surfaces of the machine shop and cable and elevator tower, will produce an interesting architectural design.

The superstructure of unit bays 1 and 2 extends above the roof of the other unit bays. The floor (El. 3244.12) at the unit bay roof level will house the cable spreading room, offices for operating personnel, and visitors' facilities. An observation deck is provided at the roof level of unit bay 3.

The top floor (El. 3256.62) will house the control room, additional offices for operating personnel, and visitors' facilities. For the benefit of visitors, the front wall of the control room will be constructed of a tubular metal frame glazed with plate glass. The ceiling of the control room will be suspended luminous plastic with lighting above. A visitors' walkway surrounding the office space on three sides will afford splendid views downstream, toward the left abutment, and the downstream face of the dam.

The working areas of the two foregoing floors will be enclosed with metal window walls, and suspended metal (with fiber glass) will be used for the ceilings. Where glazing is not required, insulated metal panel inserts will be used in the metal window walls. Movable metal partitions will be used to divide the office areas. Visitors' walkways will have protective glazed railings constructed of tubular metal frames, metal panels, and safety glass.


TON ↓
100 Ton crane
impact
impact
el one
B-GANTRY CRANE - 10 TON
9'-0" c. to c. rails
7.6 K
18.5 K
Crane wt. 21 Kips
12'-6"
18.5 K
7.6 K
C - POWER TRANSFORMER ↓
7'-6" Wheel gage
L = 17'-6"
W = 14'-0"
h = 27'-6"
Single phase
6'-0" Wheel gage
Kips per wheel incl. oil 80 K
Add 10 % impact
Wt. (incl. oil) 320 Kips
D - CABLE REEL ↓
4'-0"
5K
5K
4"
E - ROTOR AND SHAFT ↓
36'-8"
Rotor dia. = 29'-2"
Rotor and shaft
Wt = 1,150 Kips
F - H-20 TRUCK
14'-0"
20"
6'-0"
8K
32 K
AXLE LOADS
Standard A.A.S.H.O.
Add 10 % for impact
MISCELLANEOUS LOADS
Temperature variation, ±40°F on exterior surf. ±20°F on interior surf.
Wind. 20 lbs. per sq. ft of projected area.
Backfill: (horizontal pressure).
Saturated earth: 85 lbs./c.f. equiv. fluid pressure.
Dry earth: 40 lbs./c.f. equiv. fluid pressure.
Earthquake: Acceleration = 0.10 g.
Maximum tailwater, El. 3180.00
Normal tailwater, El. 3143.00
Minimum tailwater, El. 3137.00
Maximum water pressure in spiral case under water hammer conditions = 300 psi.
Pressure in spiral case during placing of conc. = 220 psi.
Hydraulic thrust plus wgt. turbine rot. pts. = 975 K. ↓
Thrust from each servo-motor = 275 Kips. ↓
Max. torque to be transmitted to the generator foundation = ± 9,859,000 ft-lbs.
G - STATOR ↓
Stator outside dia. = 35'-5"
Wt = 1,000 Kips
H - AIR CIRCUIT BREAKERS ↓
6'-8" Units 2, 4, 6, & 8
9'-8" Units 1, 3, 5, & 7
h = 13'-1"
Static wt, 19 Kips
Imp. load, 9.8 Kips
J - UPPER BEARING BRACKET
Wt 218 Kips
35'-5" O.D.
EQUIPMENT WEIGHT
12" H.P. Twin strainer 4.9K
ALLOWABLE INCREASE OF UNIT STRESSES
Dead, live, wind or normal water ..... 0 %
Dead, live, wind, and max annual water ..... 25 %
Dead, live, and temperature ..... 25 %
Dead, live, normal water, and earthquake ..... 33 %
Dead, live, normal water, earthquake, and temp. ... 50 %
Temporary and construction loads ..... 33 %
STRESSES
Reference drawing 103-D-563
Concrete: 3000 psi (ultimate at 28 days)
Reinf.: f_s = 20,000 psi in flexure
f_s = 16,000 psi web reinf.
TEMPERATURE REINFORCEMENT
Use: .005 Min. for roof and deck.
.004 Min. for superstructure walls.
.003 Min. for exterior members.
.002 Min. for interior members.
Increase percent of reinforcement adjacent to areas of restraint.

LOADS AND UNIT STRESSES

The floor in the control room will be of rubber tile, and asphalt tile will be used in the office areas and the enclosed public areas. Exterior visitors' walkways and public areas will have a concrete floor finish.

At present, it is contemplated that visitors touring through the powerplant will leave the roadway of the dam by the elevator in Block 17 and descend to El. 3187.50, thence through the adit to the downstream face of the dam. (Provisions have been made in the visitors' adits for the use of architectural finish on the floor and walls.) The dam and the powerplant are connected by two covered walkways.

Entrance to the powerplant from the walkway is provided in unit bay 1 at El. 3188.50, the balcony floor, that is 20 ft above the generator floor. From this point an unobstructed view may be had covering the entrie length of the unit bays.

Entering the cable and elevator tower at El. 3188.50 the visitor ascends by elevator to the control room floor (El. 3256.62) where he may view the control room and enjoy the vistas that are available from the visitors' walkways.

Comfort facilities are provided in the cable and elevator tower. Here again provision is made for architectural finish on floor and walls.

From the control room floor the visitor may descend by elevator or stair to the cable spreading room floor and the adjoining observation deck.

Returning to El. 3188.50 the visitor is conducted along the balcony adjacent to the downstream wall of the powerplant to the service bay area. Exit from the powerplant is provided in unit bay 8 (El. 3188.50), where a covered walkway extends to the downstream face of the dam. Proceeding through the adit in the dam to the elevator the visitor ascends to the roadway of the dam to complete the tour.

ACKNOWLEDGMENT

The writer acknowledges the assistance of the following members of his staff in the preparation of this paper: H. H. Bennet, W. H. Wolf, H. Pringle, and H. G. Kennedy. Merritt-Chapman and Scott Corporation are the prime contractors for the project.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

MODEL AND PROTOTYPE RESEARCH ON FISH LADDERS

By Berton M. MacLean,[1]

SYNOPSIS

The Corps of Engineers has been continuously engaged in the design, construction, and operation of fish passage facilities at major multipurpose water resource projects on the Columbia River for the past 25 yr. At the present time (1961) the Corps operates major fish passage installations at three large projects, has two more installations under construction and several others in various stages of planning. This paper provides a resume of the more significant fishery and hydraulic model studies that were conducted in developing the fish ladder facilities for these projects and summarizes the important operational studies conducted to determine the most efficient method of operating these facilities. To determine the responses of fish to the various hydraulic conditions that may be encountered for hydraulic model studies emphasis is placed on the need for a coordinated program of fishery research. This is to be further assured that design will provide the hydraulic conditions desired and that operational studies will make certain that the facilities are operated in a manner to provide the best conditions for fish passage within their design capability.

INTRODUCTION

The problem of passing fish over a dam or other barrier to fish migration is not new. Fishways have been built in connection with dams in Europe for

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

[1] Chf., Fish and Wildlife Sect., U. S. Army Engr. Dist., Walla Walla, Corps of Engrs., Walla Walla, Wash.

several hundred years. Early in this century G. Denil of Brussels, Belgium began a series of extensive observations and hydraulic measurements on small-scale models and compared them with actual fishways. From these studies he developed a fish ladder design now referred to as the Denil type fishway. His later research was primarily concentrated on the refinement of this specific design. Denil conducted numerous experiments on the hydraulics of fish ladders, the nature and magnitude of resistance encountered by fish in various types of ladders, and the ability of fish to overcome these resistances.

The principles developed by Denil were further investigated by the Committee on Fish Passes, British Institution of Civil Engineers at the Imperial College of Science and Technology. They made extensive hydraulic model studies on various fish ladder designs then in use throughout Europe. Their investigations were directed toward determining the dimensions and general arrangement of a fish ladder to provide efficiency as well as economy.

The advent of major water resource development projects on the Columbia River about 30 yr ago placed new emphasis on the importance of fish ladders and fish ladder design. In 1933 the Corps of Engineers was assigned the responsibility for design and construction of Bonneville Dam on the lower Columbia River, a stream supporting large runs of anadromous fish. Because the preservation of these runs was deemed essential to the economy of the region, fish passage facilities had to be developed that would be capable of passing fish runs of greater magnitude and on a larger river than had ever been encountered before. An engineer-biologist team composed of personnel from the Federal and State fishery agencies and the Corps of Engineers applied the meager knowledge then available to the task of designing suitable fish passing facilities. They found it necessary to rely heavily on the "judgment factor" when determining criteria for design of hydraulic conditions associated with fishways. Although the responses of fish to the various structures being planned could not be studied on a prototype facility, hydraulic model studies, when interpreted in the light of known responses and reactions of migratory fish, were found to provide important information regarding the success or failure of a proposed facility.

Considering the limited basic data that was available at the time of design concerning the reactions of fish to various hydraulic conditions, the fishways at Bonneville Dam have proven most successful. With minor modifications and operational adjustments within the flexibility provided for in the design, these same facilities are operating successfully today.

In 1945 the Corps of Engineers was given the responsibility for the design and construction of McNary Dam, another major project located on the Columbia River approximately 150 miles upstream from Bonneville Dam. This project also transected the magration route of anadromous fish runs in the river. Because the basic fish passage problems at the two projects were similar, the design of fishways for McNary Dam was based on the Bonneville installation with modifications provided to improve operating conditions. As little basic research concerning fish ladder requirements had been conducted during the intervening period, few proven criteria were available other than the apparent success at Bonneville. Because the efficiency of the over-all facility or its component parts had not been evaluated, the need for many of the design features and the requirements for improvement of design remained unknowns. Once more, the engineers and biologists had to rely heavily on the "judgment factor" in developing the fish facilities for McNary Dam.

Extensive model investigations were conducted in an effort to determine the probable effects of the various structures on fish magrations in order to reduce the need for, or to provide guidance in, "judgment" application. Currents in the river adjacent to the fishway entrances, flow conditions within the ladder structures, and pressures and velocities within the auxiliary water supply conduits, were of particular interest. The results of these studies assisted materially in the positioning of the entrances in relation to river flows, in alignment of downstream structures in relation to the outflow from the powerhouse units and spillway, in alleviating concern that fingerlings would be injured while passing through the auxiliary water conduit system, and in indicating the need for additional excavation downstream from the project to provide satisfactory hydraulic conditions leading to the fishway entrances.

The absence of definite criteria or standards for use in most phases of fish facility design prompted the Corps of Engineers to initiate a program of fisheries research in 1951. The basic goal was the assurance that the facilities provided at new projects were economical and adequate for the purpose intended.

PROTOTYPE OPERATIONAL STUDIES

In the design of fish facilities for McNary Dam (Fig. 1) it was the intent that operational studies would be conducted to determine the combination of entrance conditions that would provide the least interference to fish passage. Provisions were made in the design to allow a wide flexibility in the operation of these facilities. Subsequent to construction, studies were developed to test the various methods of operation, with prime consideration being given to efficient passage of fish and the elimination of any unnecessary or ineffective use of supplemental water.

Usage by Fish of the Various Fishway Entrances.—After the permanent fish collection facilities at McNary Dam were placed in operation, studies were made to obtain information concerning the relative use made by fish of the various entrances.[2,3] Fish counts were obtained at temporary counting stations placed at strategic locations in the system. An analysis of these counts indicated that the entrances across the downstream face of the powerhouse collected five times as many fish as the large south shore entrance; the entrances near the center of the powerhouse collected few fish; and the number of fish entering the large entrance between the powerhouse and the spillway (non-overflow entrance) was negligible even during periods of no spill.

The information developed from this study indicates that fishway operation and future designs should concentrate on improving the hydraulic conditions leading to the powerhouse collection system and de-emphasize the importance of the so-called major entrances located at either end of the powerhouse. The design of facilities for projects now under construction are based on these findings. In recognition of the importance of entrance conditions, extensive model studies have been conducted to evaluate specific problems. The model studies not only provide guidance in achieving improved fish attraction conditions but also have demonstrated means of conserving attraction water.

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2 "Travel Rate of Chinook Salmon in Oregon Fish Ladder at McNary Dam," U. S. Army Engr. Dist., Corps of Engrs., Walla Walla, Wash.

3 "Fish Usage of Collection Facilities at McNary Dam," U. S. Army Engr. Dist., Corps of Engrs., Walla Walla, Wash. Unpublished.

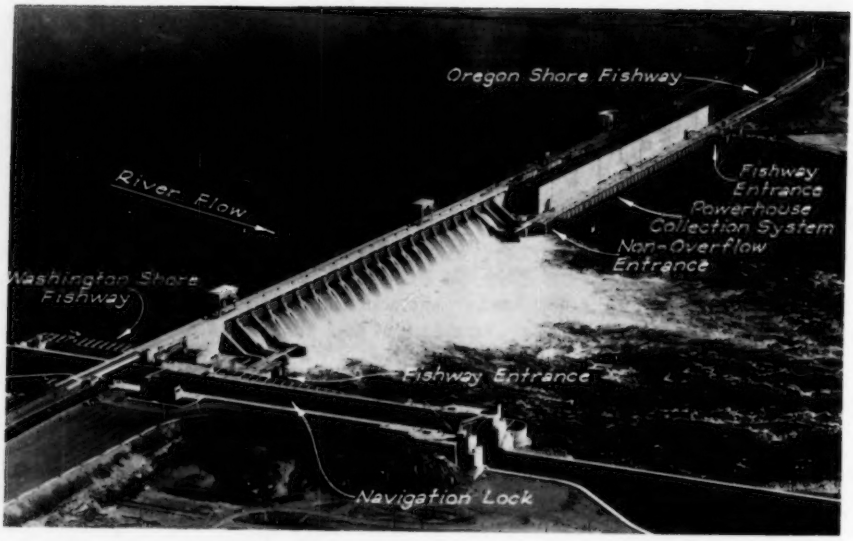

FIG. 1.—McNARY LOCK AND DAM

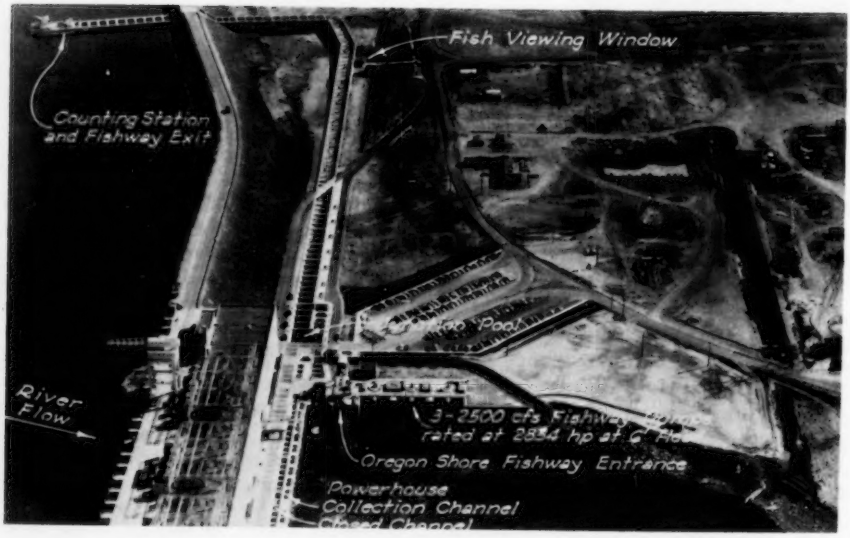

FIG. 2.—McNARY DAM SOUTH SHORE FISH ENTRANCE

McNary South Fish Entrance.—At McNary, considerable study has been centered around the entrance conditions present at the south shore immediately downstream from the powerhouse (Fig. 2). In this area the design provided for three overflow entrance weirs per power unit located across the downstream face of the powerhouse, each having an attraction flow of 60 cfs. Along the shore immediately downstream from the powerhouse, a major fish entrance consisting of two bays each 15 ft in width was provided. The flow from this entrance entered the river at an angle of 60° to the tailrace flow. It was designed to discharge a large quantity of water that would readily attract fish into it.

As information became available on the responses of fish to flow conditions, the need for a major entrance adjacent to the powerhouse collection system was questioned. A number of experiments were planned to evaluate the influence of this entrance in attracting and passing fish over the dam.[4,5] Each experiment consisted of operating the fish facilities at the dam for a given period of time with the south entrance open, followed by a similar period with the entrance closed. Fish passing through the ladders were recorded for all test periods. These cyclic operations were conducted during the fish migration seasons of 1955, 1957, and 1958. A detailed statistical analysis of the data indicated that the elimination of the south shore entrance had no significant effect on passage of fish at the project.

Rate of Fish Travel in Fish Ladder.—Another item concerning fish passage that was investigated at McNary Dam was a determination of the time required for fish to travel through the fish ladders. Basic data was collected by counting all fish passing selected fixed weirs, as well as at the counting station at the upper end of the ladder. An analysis of these counts indicated that the average time fish required to move through each ladder pool was 2.6 min and 2.4 min for spring chinook and blueback salmon, respectively.[2]

In addition to travel time information, fish were found to stop moving into the ladder and at the counting station at the upper end of the ladder immediately after dark. However, those fish already started up the ladder did not stay in the pools during the night but continued up the ladder until slack water was encountered immediately below the counting station. These fish moved through the counting station at daybreak the following morning.

Very few "dropbacks" or fish moving downstream were recorded during these tests, thus giving support to the theory that swimming through a McNary type ladder is only a mild exercise for fish.

Submerged Orifice Entrance.—In 1948, Corps of Engineers personnel developed considerable evidence that entrances equipped with submerged orifices in the powerhouse collection system at Bonneville Dam were more effective in collecting fish than were entrances with overflow weirs. During 1952 and 1954, a series of studies was conducted to determine, under prototype conditions, the relative efficiency of a submerged orifice entrance as compared with an overflow entrance utilizing the same quantity of water. The 1952 experiments con-

4 "The Influence of the South Shore Entrance on the Oregon Shore Fish Collection System in Passing Fish Over McNary Dam," by Frederick K. Cramer, Berton M. MacLean, and Frederick A. Davidson, U. S. Army Engr. Dist., Corps of Engrs., Walla Walla, Wash., 1958.

5 "Supplementary Report on the Influence of the South Shore Entrance on the Oregon Shore Fish Collection System in Passing Fish Over McNary Dam," by Frederick K. Cramer, Berton M. MacLean, and Raymond C. Oligher, U. S. Army Engr. Dist., Corps of Engrs., Walla Walla, Wash., 1959.

sisted of comparing the operation of a shallow submerged orifice with an overflow weir in successive alternating periods. The 1954 experiments continued the submerged orifice versus overflow weir experiments with added phases on comparison of efficiency of orifices at various depths of submergence.

The resulting data indicated that a submerged orifice set from 2 ft to 8 ft below tail water elevation provided the most attractive entrance condition.[6]

Based on the results of these studies, the powerhouse collecting system entrances at McNary and Dalles Dams were changed from overflow to submerged orifice operation. The design for similar entrances at projects now under construction include provisions for submerged orifices in self-adjusting buoyant gates.

FISH LADDER LABORATORY AND MODEL STUDIES

As part of the Corps of Engineers Fisheries Research program, a special laboratory costing more than $500,000 was constructed to study the behavior and performance of migrating fish under a wide variety of fishway situations. The laboratory, located at Bonneville Dam on the Columbia River, is now in its fifth year of operation. The research is being conducted by the U.S. Bureau of Commercial Fisheries under contract to the Corps of Engineers. Highly significant progress has been made in several important categories of the fishway problems.

Fishway Capacity Studies.—Probably the most important items of research undertaken to date are studies to determine how many fish will pass through a fish ladder of a specific design. Examination of data collected on rates of movement shows that, although rates vary with species and time of year, there is considerable consistency in the average rates of movement under a wide range of experimental conditions. During one specific experiment, salmon were passed through a 1 on 16 slope fishway only 4 ft wide at a rate of 3,000 fish per hr without any indication that passage capacity had been reached.[6,7]

Fish Ladder Length and Slope.—At dams on the Columbia River, fish ladders have been built with slopes ranging from 1 on 10 to 1 on 20. As far as can be determined by observations, all of these ladders are providing satisfactory passage for fish. Some biologists were of the opinion that fish use more energy in climbing a ladder of 1 on 10 slope than one with a slope of 1 on 20. If this premise were correct, fish, in passing several dams, might require enough additional energy to adversely affect their ability to complete the spawning process. This factor and the monetary savings that would be realized by using the steeper slope design prompted a study to evaluate the significance of slope on fish movement.

Initially, two short 11.5-ft-wide fish ladders were constructed, one with a slope of 1 on 8 and the other with a slope of 1 on 16. Experiments comparing the performance of salmonoids in these fish ladders indicated a faster rate of passage in the steeper slope ladder.

Additional experiments were undertaken using a pair of "endless" fish ladders with slopes of 1 on 8 and 1 on 16. Each ladder was 3 ft wide and consisted of 16 pools. The highest and lowest pools were connected by a fish lock, thus

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6 "Progress Report on Fisheries Engineering Research Program," U.S. Army Engr. Div., North Pacific, Corps of Engrs., Portland, Oreg., 1960.

7 "Fishway Research at the Fisheries - Engineering Research Laboratory," U. S. Fish and Wildlife Service, Circular No. 98, 1961.

making a complete circuit. As the fish reached the top pool, they were rapidly lowered to the bottom pool, there to begin the ascent again. By continuing this procedure, a fish ladder of any desired length could be simulated. Most fish were tested with a climb of approximately 100 pools, although a limited number were permitted to travel several hundred pools. Four from each species were allowed to climb over 1,000 pools. One blueback swam through 6,600 pools before the test was terminated. This constituted a vertical height of over 1 mile in a 1 on 8 slope fish ladder.

With performance as the main basis of comparison, the results of these experiments indicate differences in patterns and rates of movement. However, no evidence of fatigue was found in fish using either fish ladder. Biochemical indices of fatigue such as lactate levels and inorganic phosphates of the blood were also used as a check on the performance methods of comparison. It was concluded that ascent in a properly designed fish ladder was only a moderate exercise for fish; the steeper 1 on 8 slope fish ladder was equally efficient and suitable for passage of salmonoids as the 1 on 16 slope, providing proper hydraulic conditions are obtained; the rate of climb in a 1 on 8 slope fish ladder of proper hydraulic design is approximately the same as in the 1 on 16 slope ladder; and the rate of fish movement in a fish ladder is independent of the number of fish present.

Based on these results, and taking into consideration that the studies were performed on a ladder 3 ft in width with solid overflow weirs, the Walla Walla District initiated hydraulic model studies on a fish ladder 16 ft in width with a slope of 1 on 10. The objective was to develop conditions within the pools that were equal to or better than those in a fish ladder with a slope of 1 on 16 and conventional overflow weirs with two submerged orifices in each weir.

Immediately there developed the problem of how to compare or evaluate the hydraulic conditions in separate pools. The two items considered significant were satisfactory conditions to induce fish to move through the pools and the availability of suitable resting areas within the pools. Percentage of total water volume having velocities of 1 fps or less was accepted as a basis for comparison of ladders, provided that suitable flow conditions from the orifice jets were maintained.

The objective was to develop hydraulic conditions within the ladder pools equal to or better then those present in a ladder in which satisfactory fish passage was known. Hydraulic model data for a 1 on 16 slope fish ladder with overflow weirs having 2 orifices in each weir was used as a base for comparative purposes. A reduction of the length of the overflow weir crest and appropriate sizing of the orifices were found necessary if conditions within the pool of the 1 on 10 slope ladder were to provide the required percentage of low velocity area. Several trials were run using various lengths and positions of the blocked-out weir section. The position of the orifice in relation to the overflow weir section was found to be an important factor. The combination that appeared to have most promise included 5-ft-wide overflow sections along each ladder wall; a 6-ft-wide non-overflow section located in the center of the weir; and two 18-in.-sq orifices flush with the floor, one located at each end of the weir with center line 3 ft from the ladder wall.[8] To stabilize the flow pattern of the jets emerging from the orifices, two vertical extensions or fins were installed

[8] "Ice Harbor Dam Fishway Model Studies," Bonneville Hydr. Lab., U. S. Army Engr. Dist., Corps of Engrs., Portland, Oreg. Unpublished.

on the upstream face of the weir, one located at each end of the non-overflow section.

This plan was the basis for design of the north shore fish ladder at Ice Harbor Dam on the Snake River (Fig. 3). In order that it could be used for future testing of a prototype ladder of reduced width, effort was made to obtain symmetrical flow conditions in the pools. This allowed the inclusion of a divider wall down the center of the ladder, making in effect two ladders, each 8 ft in width.

Subsequent to design, 6 pools of this ladder were built to full scale in the fisheries laboratory and tested during the fish runs of 1960. The facility operated very successfully. The average time required for fish to travel through the pools was not materially different from other ladders tested. Performance of fish was found to be substantially the same in the 8-ft-wide ladder (divided)

FIG. 3.—FULL SCALE ICE HARBOR 1 ON 10 SLOPE FISH LADDER MODEL IN LABORATORY

as in the 16-ft-wide ladder. Plans are being developed to make extensive observations and experiments in the completed ladder during 1962, the first year of operation of the fish ladders at Ice Harbor Dam.

Additional studies concerning the adoption of this plan for use in the design of fish ladders for John Day Dam are now in progress.

Fish Attraction.—Another item of major importance to fishway design investigated under the Corps' fishery research program was the water velocity most suitable for a fishway entrance. Two criteria were utilized in the experimentation: the swimming ability of fish and the response of fish to choice of velocities. In the swimming ability category a flume 85 ft long and 5 ft in width, in which water velocities could be varied, was used as a testing device.

A definite difference in performance was found both with respect to species and also with respect to size within a species. Large fish of the same species were found more capable of superior performance than small fish. Maximum swimming speeds in relation to the water were recorded as 26.7 fps for steelhead trout and 22.1 fps for chinook salmon when measured over a distance of 30 ft.[7] These species registered approximately equal median rates of movement in water of 8 fps velocity.

Two entrances, each 5 ft in width and utilizing different velocity rates through each entrance, were used to test the fish in the choice of velocity experiments. Fish were allowed to choose between a high and a low velocity flow. In every case a greater percentage of the fish tested entered the channel of higher velocity flow. This choice prevailed even under conditions in which fish could not negotiate the channel for any appreciable distance after entry. Many fish that were swept back after failing to pass through the flume selected the higher velocity entrance on their second attempt.

Fish Ladder Water Control.—Another critical item in fish ladder design in which hydraulic models have played an important role is the method for water control at the upper end of the ladder. A fish ladder requires a constant quantity of water if the desired hydraulic conditions within the ladder are to be maintained. When the upper end of a fish ladder terminates in a fluctuating reservoir, some means of controlling the quantity of water entering the fish ladder and a means of providing suitable ladder pool conditions as the forebay fluctuates must be provided. At Bonneville Dam the forebay may fluctuate as much as 10 ft. In the original design, flow control was maintained by means of a regulating gate at the upper end of the ladder. Continuation of pool steps was provided by means of removable stop-log weirs. As the reservoir fluctuated, ladder pools were added or deleted by insertion or removal of these stop-log weirs.[9] This proved to be a cumbersome procedure with the hydraulic conditions within the ladder pools constantly changing. The upper weirs were modified to orifice bulkheads that provided suitable control for minor forebay fluctuations.

Having experienced the operational problems associated with the Bonneville design, the Corps developed a series of automatically operated tilting weirs for the McNary Project to provide the necessary continuation of the pool steps and a regulating weir to provide the needed quantitative control of the flow for the varying reservoir elevations (Fig. 4).[10] The regulating and tilting weirs function as a unit to provide the desired flow conditions. The tilting weirs are hinged at the bottom and may be rotated through an arc of 90°, horizontal to vertical. The most significant features of this system are that it provides adequate control of the flow and operates automatically for all changes in reservoir elevation. As the reservoir level fluctuates, the regulating weir moves to retain its same respective depth below the pool; and each upstream tilting weir, in turn, also moves to retain a uniform differential between each successive pool.

Although the tilting weir method of control at the upper end of McNary Dam fish ladders operated most successfully, it was expensive and required considerable attention and maintenance. In considering the facilities for Ice Harbor Dam, the Corps investigated the possibility of developing a control section

9 "Information Bulletin, Fish Passage Facilities, Bonneville Dam," Report No. 66-1, U. S. Army Engr. Dist., Corps of Engrs., Portland, Oreg., 1958.

10 "Fish Facilities for McNary Dam," Technical Report No. 30-1, Bonneville Hydr. Lab., U. S. Army Engr. Dist., Corps of Engrs., Portland, Oreg., 1960.

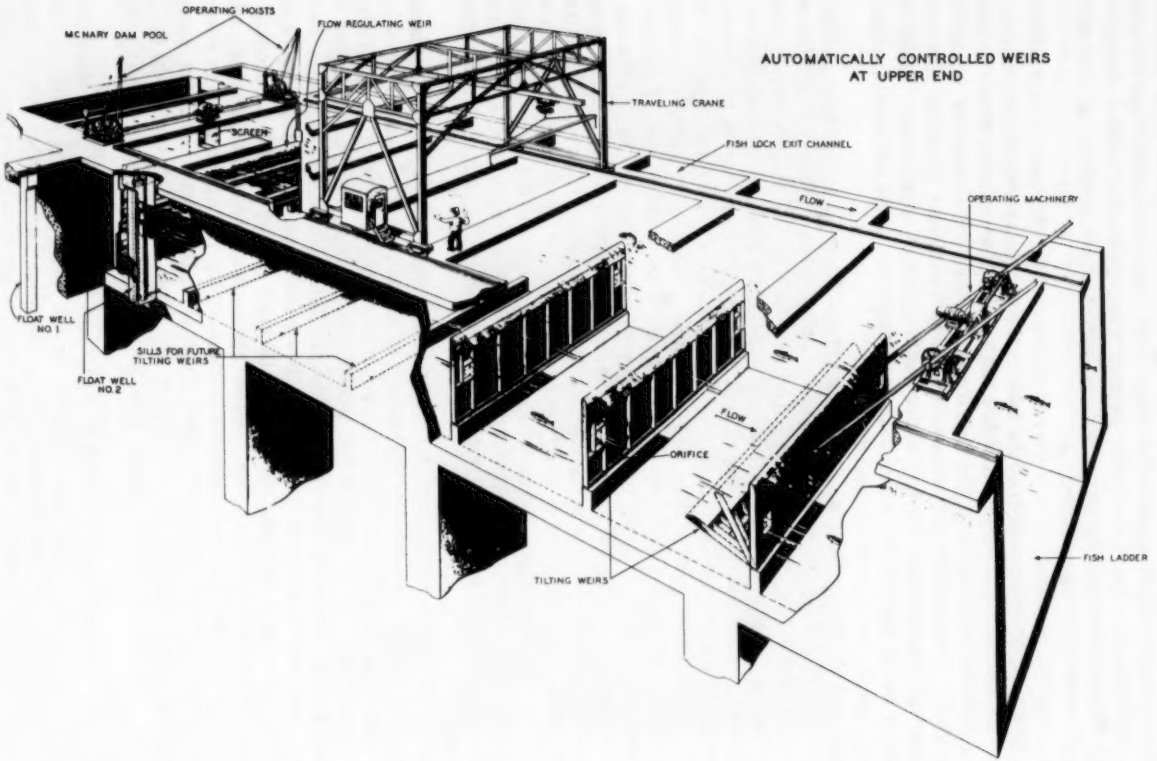

FIG. 4.—McNARY DAM FISH LADDER CONTROL SECTION

consisting of a series of non-overflow submerged orifice weirs. An extensive model study was conducted to determine the length of pools and size and arrangement of orifices required to provide the desired hydraulic conditions within the pools.[8] The final design of this orifice control section includes a series of transverse bulkheads, each equipped with two orifices (Fig. 5). As the pool fluctuates, the flow through this section varies. The drop at each bulkhead varies correspondingly. The balance of water to make up the required

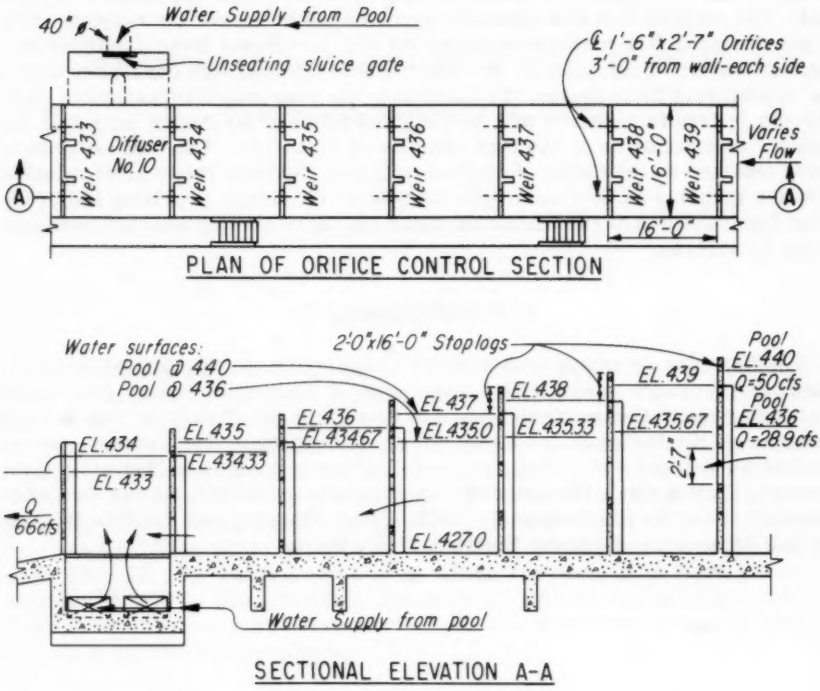

FIG. 5.—ICE HARBOR DAM FISH LADDER CONTROL SECTION

ladder flow is added by means of an automatically controlled sluice gate. The water from this gate enters the ladder floor immediately below the control section.

CONCLUSIONS

The results of fishery research conducted or sponsored by the Corps of Engineers have been most gratifying. Considerable improvement in operational efficiency has been made, and several millions of dollars have been saved in the fishways at projects now under construction by the Corps and other public and private organizations without sacrifice of performance. Our experience

indicates that a closely coordinated program encompassing fishery research and hydraulic studies, using both prototype and model facilities, is not only desirable but essential in developing improvements in operation and economics in design of fish ladders and appurtenant facilities. Each species of fish has its own particular characteristics and abilities. Fishway design must take these factors into consideration if efficient designs are to be obtained. Continued expansion in water resource development will require an accelerated program in determining the capabilities and responses of the various species of fish to the multiplicity of hydraulic conditions they may encounter at a project. The construction and operating agencies not only have the responsibility to provide good fish passage facilities but also to operate these facilities in a manner that will economically provide the best fish passage conditions within the capability of their design. To discharge this responsibility, extensive fishery and hydraulic research will be required not only for design purposes but also for determination of the best methods of operation. The vast experience being obtained by the Corps of Engineers in this field will prove most valuable to other agencies as they undertake the task of efficiently providing and operating fish facilities at the numerous water resource development projects now being considered.

ACKNOWLEDGMENTS

The fisheries experiments described and resulting data presented were obtained by research conducted by the Corps of Engineers and by the United States Bureau of Commerical Fisheries as part of the Fisheries Engineering Program of the United States Army Corps of Engineers. The hydraulic model studies were conducted by the Corps of Engineers Bonneville Hydraulic Laboratory. During the preparation of this paper, many helpful suggestions were made by Glenn H. Von Gunten, F. ASCE, Chief, Planning and Reports Branch, United States Army Engineer District, Walla Walla.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

PRIVATE POWER DEVELOPMENT OF AN ENTIRE RIVER[a]

By Richard S. Woodruff,[1] M. ASCE

SYNOPSIS

The development of a complete river by one company for hydroelectric power production is described. Construction is underway (in 1961) on the Coosa River in Alabama, and the development program is coordinated with an act of the Congress of the United States authorizing the work with the inclusion of certain required provisions for flood control and future navigation.

In 1957 the Federal Power Commission (FPC) issued a single license providing for the construction of five projects that must be completed within a 10-yr period. This paper presents a history and description of the Alabama Power Company, the basic studies made within the basin area leading to the development of final plans, brief statistics for all present and proposed power plants on this river, general design criterion, and the present status and magnitude of construction operations.

INTRODUCTION

On the 28th of June, 1954, Dwight D. Eisenhower signed a bill that became Public Law 436 of the Eighty-third Congress. This law authorized the Alabama Power Company to develop the Coosa River. Prior to this time, the Corps of Engineers had been authorized to develop this river for flood control, navigation, and power by the Rivers and Harbors Act of March, 1945. Public

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

[a] Presented at the March, 1960 ASCE Convention at New Orleans, La.

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Law 436 removed the power feature of the 1945 act and provides for a partnership endeavor; the company is to economically develop electric power by a series of dams in accordance with the conditions of a license, if issued, pursuant to the Federal Power Act, and the government would develop the other features as later warranted.

This law requires the development substantially in accordance with the Corps of Engineers' plans. It establishes a reasonable time limit for completing the entire development and sets up "recapture" provisions for non-fulfillment of conditions. No provision was made for government financial assistance for construction of the projects.

The act providing for the development of the Coosa River by private enterprise has certain pertinent points that are incorporated in one Federal Power Commission license, issued to the company in September, 1957.

COOSA RIVER BASIN

The Coosa River is 286 miles long and is a part of a major drainage system in the Southeastern United States. It is formed at Rome, Georgia, by the junction of two northwest Georgia streams and extends through north central Alabama to a location near the city of Montgomery, Ala., where it joins the Tallapoosa River to form the Alabama River. This, in turn, flows south through Alabama and empties into the Gulf of Mexico at Mobile, Ala. (See Fig. 1).

The Coosa River has a total fall of approximately 450 ft. Of this total, 230 ft in the lower 70 mile reach had previously been developed by the company with three plants constructed 35 yr to 45 yr ago. Its average flow is 16,000 cfs with a record peak discharge of 298,000 cfs at Wetumpka, Ala. near its mouth.

The drainage area of the Coosa basin is approximately 10,300 sq miles. It is located in an area of moderate climate with an average annual rainfall of 53 in. The basin lies in the southern foothills of the Appalachian mountains and its gradient is quite uniform and without extensive erosion. Geologically, the area has extensive faulting with sandstone and limestone predominating. Shales are prevalent throughout the valley and the Coosa coal field, not economically productive, parallels the central valley. In general, foundation conditions are excellent for any type of concrete or earth structures.

GENERAL COMPANY DESCRIPTION

Size and Load Growth.—The Alabama Power Company, with its main office in Birmingham, Ala., has an operating area that covers most of the state of Alabama. The company was organized in 1906 to generate electricity by means of water power, and in its early development several dam sites were acquired on the rivers of Alabama including the Muscle Shoals site on the Tennessee River. Presently the company has a total of thirteen existing power plants, six of which are hydro; however, approximately 80% of the total kilowatt hours generated is by steam. The completion of the Coosa River development will add approximately 425,000 kw of hydro capacity to the company's system, which at the beginning of 1960, consisted of 489,700 kw of hydro power and 1,436,250 kw of steam power.

Industrial growth and new uses for electrical energy since World War II have resulted in a load growth of about 8.2% per yr on the Alabama Power sys-

FIG. 1.—ALABAMA POWER DEVELOPMENTS

tem. This means the company must double its generating capacity approximately every 10 yr. The majority of this capacity will come from coal fired steam generating units. After two new steam plants had been built and additions made to others, it was decided in 1953 to initiate investigations of additional sources of hydroelectric power. The location of the Coosa River was such that its entire power potential could be readily absorbed within the company's system, as it is located in close proximity to rapidly growing industrial areas. The generally increasing costs of fuel, both coal and gas, also make it desirable to develop sources of hydro power.

Early River History.—The development of the Coosa River has been under consideration since 1870. Under various congressional authorizations, six government projects for navigation had been completed on the Coosa River by 1918. Their continued operation was not justified and in 1920 the Chief of Engineers reported to the Secretary of War his opinion that the whole existing project should be abandoned, due to lack of commercial use. The upper reach of the river around the turn of the century, had some commercial steamboat activity between Gadsden, Alabama and Rome, Georgia. Prior to the Civil War (pre 1860's) there were paddle wheelers operating on the Alabama River from Mobile to the rapids at Wetumpka, on the Coosa River. One of the reasons for the organization and founding of the Alabama Power Company was to further develop the Coosa River for navigation by the construction of private power dams. The first major power development in Alabama was the construction of Lay Dam by the Alabama Power Company, on the Coosa River near Clanton, Alabama in 1914.[2]

The increasing electric power requirements in Alabama following World War I made it necessary to construct additional power producing facilities. Two more hydro electric plants on the Coosa River were constructed in the 1920's. Mitchell Dam, downstream from Lay Dam, was finished in 1923 and Jordan Dam, below Mitchell, was finished in 1928 (See Fig. 2).

PRELIMINARY INVESTIGATIONS

Recent Planning.—Late in 1953, extensive investigations to develop the best series of dams for the remaining undeveloped portions of the Coosa River were initiated. These studies extended over a period of approximately 4 yr. However, in the latter part of 1955, studies had progressed to such an extent that an application for a license to develop the remainder of the Coosa River was made to the Federal Power Commission.

Mapping.—The undeveloped portion of the Coosa River was mapped by aerial photography with approximately a quarter million acres mapped under top contours. These maps were made with 10-ft contour intervals between Lay Dam and Gadsden, Ala., and 5-ft contour intervals between Gadsden and Rome, as this portion of the state was less hilly. Pictures were made in March, 1954 by flying at 12,000 ft and 6,000 ft, respectively. The map work was done on a scale of 1 in. equal to 1,000 ft, requiring 84 sheets 27 in.-by-37 in. The ground control for mapping was run by the company's personnel. At least three picture points per photo were established for elevation. This required some 650

2 "The Lock 12 Development of the Alabama Power Company, Coosa River, Alabama," by E. L. Sayers and A. C. Polk, Transactions, ASCE, Vol. LXXVIII, 1915, p. 1409.

FIG. 2

miles of second order traverse, 900 miles of second order levels as well as 1,850 miles of third order traverse and 1,150 miles of third order levels. The accuracy of the maps was guaranteed to be within one-half of one contour interval. In the interest of time and economy, contours were not plotted above established maximums within the ranges of possible reservoir areas and flood flow lines. Field surveys were made of all bridge and utility river crossings. These maps were very successful and are widely used. Time, as well as lack of experienced personnel for plane table work, did not permit field mapping, and the maps that were available were not suitable for the company's use.

Drilling.—The geological and foundation investigations involved drilling over 3,000 core holes from which some seven miles of cores were obtained. Approximately the same number of auger holes were drilled through the overburden to determine thickness, nature of material, and the elevation of the rock surface throughout the basin. The core holes were drilled to a depth of 75 ft below top of rock, although a few holes were drilled about 300 ft deep for a better understanding of subsurface characteristics. Drilling was done by contract under the company's supervision with as many as eight rigs in operation at one time. Representative samples of different core types were subjected to physical and chemical tests for analyses.

Auger holes were drilled with 4 in. augers in order to obtain a thorough sampling of the overburden materials and to penetrate to approximately 40-ft depths. Selected materials were bagged and sent to a centrally located laboratory for size gradation analyses, maximum density, optimum moisture, and permeability tests. Samples from possible borrow pit and earth dike locations were also sent to a commercial laboratory for tri-axial shear tests.

Geology.—The upper Coosa basin, Rome to below Gadsden, is in very tight shale that contains varying amounts of thin limestone beds. When it is exposed to air for a few hours it disintegrates badly. Several sites were investigated in this formation and in some of these the shale was so soft and friable that it could not be cored. Generally these sites contained less calcareous material than the one finally chosen. The final site had near vertical limestone beds, up to 2 ft thick, in which the bedding had been up-ended by a thrust fault at close proximity.

The Gadsden-Lock 3 area is underlain by various shale formations with some beds of sandstone; however, the Lock 3 dam site (temporarily identified by nearness of old government Lock 3 navigation dam) is located on limestone that is somewhat cavernous.

For the area from Lock 3 to Lay Reservoir, the Coosa River flows through a limestone and dolomite area. This rock is typically cavernous resulting in some underground drainage. Some cavities are completely filled with varying proportions of clay and sand and gravel; others are only partly filled, and some are empty. The rock surface is saw-toothed and irregular. In any plan of development it was necessary that one dam be located in this region.

The lower part of the river system, Lay Reservoir to Wetumpka, is underlain by hard mica schist that contains lenticular quartzite dikes. Below Wetumpka the surface of this schist dips to the south beneath younger unconsolidated clayey sand and gravel of Cretaceous age.

At some of the more promising dam sites, test grouting was done to develop additional information on the character of the rock, but this did not disclose any unusual or unexpected conditions. The electrical resistivity method of investigating subsurface conditions was tried in a dolomite area in which cavities

were known to exist from prior core drilling; however, the results were not conclusive. The quality of river water is also investigated in order to determine its condition prior to impoundment.

Supervision.—The investigative work and general planning was done under the direction of F. C. Weiss, Vice President, Engineering and Construction. Periodic staff meetings were held to discuss the field work and engineering planning, with specialists invited from time to time. These meetings were recorded and the tapes have been used to review discussions. Planning was frequently discussed with various state officials because the scope of this project is statewide.

Exploration Cost.—The Alabama Power Company spent approximately two million dollars on the preceding exploration endeavors. Although there was no doubt of the feasibility of developing the entire river, the exploratory work was necessary to assist in determining the most economical method with the greatest benefits.

DEVELOPMENT PLAN

Federal Power Commission License.—After intensively studying some twenty-seven different plans of development over a 2 yr period of time (1954 to 1955), a scheme for the most beneficial plan was developed. An application for a Federal Power Commission (FPC) License was filed in Washington, D.C. on December 2, 1955. Although this application was pending, studies were continued and further advanced. On September 4, 1957, the company received a 50-yr license for the Coosa River Development known as Project No. 2146. This covers four new dams with power plants and the modification of an existing dam and power plant.

The license requires that the construction sequence of the projects be related to their flood control importance, that the first project be started within 1 yr, and that all projects be completed with initial power installation within 10 yr from the beginning of construction of the first project.

A profile of the Coosa River with the development plan and its relation to existing hydro plants is shown in Fig. 3, and a tabulation of the main characteristics of these plants is shown in Table 1.

The Weiss Dam project, Fig. 4, is somewhat unusual. The powerhouse is located approximately 4 miles away from the spillway. It is connected to the reservoir by a forebay lock and a canal. This takes advantage of the head in a by-passed 20 mile reach of the river.

The Lock Three, Logan Martin (formerly called Kelly Creek), and Wetumpka Dams are not unusual. The powerhouse will be adjacent to the spillway for each of these projects, and they will resemble the Weiss spillway and powerhouse in appearance, except for the Logan Martin Project for which advanced studies have shown the advantages of changing to an outdoor type powerhouse.

Lay Dam (Fig. 2) will be raised 14 ft, and a second powerhouse will be constructed adjacent to the present one. This dam was constructed prior to passage of the Federal Power Act, so it is not under FPC license. The present license, however, will now include Lay Dam after it has been modified. Mitchell and Jordan dams are both FPC licensed projects. Planning studies are, of course, continuing, looking towards additional refinement of the projects still to be constructed.

TABLE 1.—COOSA RIVER HYDRO PLANT DATA

Project Name	Gross Head, Feet	New Installations		Existing Installations		Reservoir Area, Acres	Flood Storage, Acre-feet	No. of Spillway Gates	Spillway Gate Size W x H, in feet
		Capacity, in killowatts	No. of Units	Capacity, in killowatts	No. of Units				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Weiss Dam	59	87,750	3	-	-	31,700	379,000	5	40 x 38
Lock #3 Dam	40	72,900	3	-	-	9,500	83,000	5	40 x 38
Logan Martin Dam	69	128,250	3	-	-	15,870	233,600	6	40 x 38
Lay Dam	84	96,000	2	81,000	6	11,700	-	26	30 x 18
Mitchell Dam	67	-	-	72,500	4	5,900	-	26	30 x 15
Jordan Dam	90	-	-	100,000	4	4,900	-	17[a]	30 x 18
Wetumpka Dam	34	44,000	2	-	-	1,200	-	10	40 x 49

[a] Approximately half of this spillway is ungated.

Cost Estimating.—All cost estimating was done from preliminary plans. To assist in preparing the best possible estimates, well known interested contractors were asked to prepare detailed estimates for a specific project. They were compensated for their work, and most of them visited the sites. Road damages were determined for each reservoir through discussions with the various County and State road officials. The cost of the completed plan for the new installations listed in Table 1 is approximately 132 million dollars.

Reservoir.—The acquisition of reservoir lands and relocation of facilities within their boundaries present the usual problems. A policy has been established to purchase land in fee to an elevation 1 ft above normal power pool level and to purchase flood rights to a higher elevation approximately equivalent to the 5-yr flood profile. It is not planned to make a detailed land survey of the reservoirs, except in a few critical areas. Relocation or removal of roads, utilities, and municipal works will be done by the agencies involved. Cemeteries are being moved by contract.

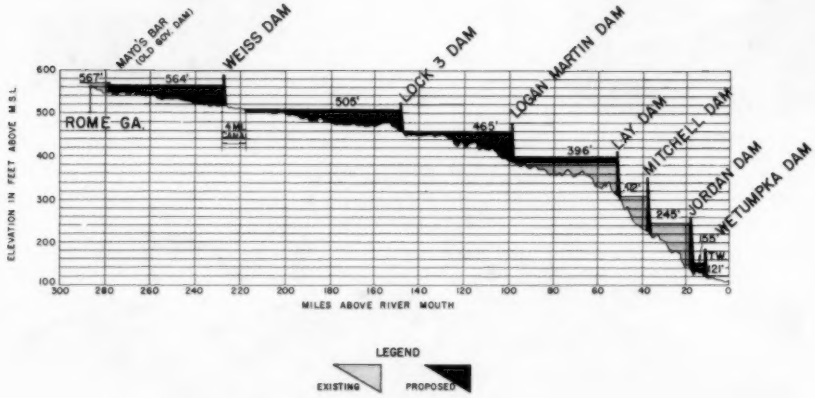

FIG. 3.—PROFILE OF COOSA RIVER

Cooperation.—In preparing development plans the Alabama State Board of Health and the Georgia Department of Health were consulted as well as the Fish and Game Commissions of the two states, because the uppermost reservoir extends into the State of Georgia. An agreement was entered into with the Atlanta Regional Office of the United States Fish & Wildlife Service to investigate the fish and wildlife potentials and develop a resource use plan to make the Weiss (largest) reservoir of maximum recreational benefit. As plans were advanced, the regional office of the FPC in Atlanta, Ga., and the district office of the Corps of Engineers in Mobile were kept advised.

GENERAL DESIGN CRITERIA

Introduction.—In order to be consistent in the design of related projects on the same river, it was necessary to establish in advance, certain general de-

sign criteria relating to project use, flood control, detail design, and safety features.

Design Storm.—A theoretical design storm producing maximum flood conditions was developed from studies of several historical storms that conceivably might occur over the Coosa River basin. The storm of March 11 through March 16, 1929, with principal center in the vicinity of Elba, Alabama, was found to have the most critical depth-duration-area relationship. This storm with its moisture content adjusted to conform with maximum recorded dewpoint temperature was positioned over the Coosa River basin (as shown in Fig. 5) to yield maximum average depths on drainage areas above the upper three dam sites. Following adjustments of the storm depths to account for elevation

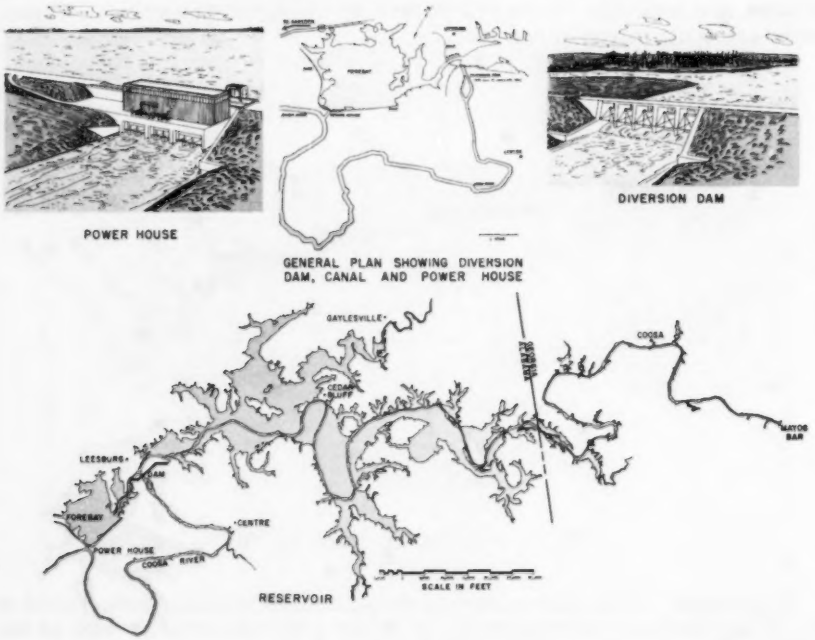

FIG. 4.—WEISS DAM

of inflow barrier and distance inland, the average depths of rainfall excess over each of the several sub-basins above each dam site were computed assuming probable minimum rates of infiltration. Hydrographs developed by applying these depths of rainfall excess to unit graphs were routed through various spillway sections to obtain the lowest combined cost of dam and spillway at each site. The inflow hydrograph to each reservoir below Weiss dam site was modified to reflect the effects of upstream storage. The maximum rate of outflow from each of the upper three reservoirs was considerably less than the estimated natural peak discharge at the respective sites. For dam sites below Kelly Creek, the storm repositioned and the procedure repeated. This storm

FIG. 5.—SPILLWAY DESIGN STORM, COOSA RIVER BASIN

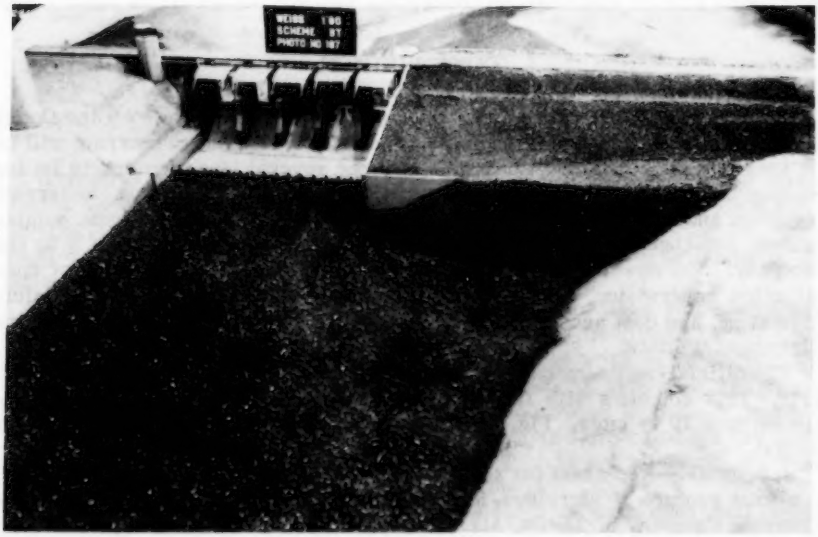

FIG. 6.—WEISS DAM MODEL

produces a flow approximately four to five times greater than the flood of record for the Coosa Valley. All project structures are designed with a minimum freeboard of 5 ft. The spillway capacities are reviewed by the Corps of Engineers and the FPC.

An independent study developed a design storm for the Coosa River basin that was essentially the storm described here.

Stability.—For dam and powerhouse design, an assumption of 100% uplift over 100% of the base area was used. The design of the earth section of dams is based on obtaining 95% compaction (modified proctor). An earthquake factor of 0.1 g will also be used for all five dams. A rock shear friction test was made at the location of the first dam by a series of concrete blocks poured on base rock with loading applied vertically and horizontally by jacks.

Miscellaneous.—Based on river flows and power requirements, a plant load factor of approximately 13.5% was established.

The proposed projects will have substantial flood control features that private industry must incorporate in the development plan to meet the requirements of the law and license.

All power plants are to be designed for semi-automatic operation with provision for future operation by remote control. Similar design and operating features are to be incorporated in all of them, including provision for a public highway crossing. An economical size spillway gate (40 ft wide-by-38 ft high) has been designed for the Weiss project, and it is planned to use the same size gate at all the new downstream plants.

Model Studies.—Hydraulic model tests are made in the University of Alabama's hydraulic laboratory. For the first dam model tests were run to determine spillway discharge coefficients, the design of a stilling basin apron, possible river bed erosion conditions, and to determine the height of cofferdam required to protect against the flood of record (See Fig. 6). Laboratory operations will be continued in connection with the design of the other dams.

CONSTRUCTION

Contracts.—The construction of each of the various projects on the Coosa River is to be done by means of several contracts. One large contract will be let for the main structures of each project and several lesser contracts let for equipment installation, powerhouse structural erection, painting, reservoir clearing, and so forth. All items of equipment, turbines, generators, pumps, cranes, switchgear, transformers, and so forth, are purchased directly by the company. The power company will set up its own field organization for construction supervision, inspection, and job accounting. General supervision, expediting, and cost accounting for all construction projects is handled in the Birmingham office.

The official beginning date for the Coosa Development project was July 31, 1958. This complies with license requirements and also establishes a completion date 10 yr later. Fig. 7 shows the construction schedule for the 10-yr period.

Job Status.—A contract for the first dam (Weiss Dam) was let in July, 1958, to a joint venture of Morrison-Knudsen Company of Boise, Idaho, and Moss-Thornton Company of Leeds, Ala. This job consists of moving approximately 6.25 million cu yd of earth and rock, pouring 150,000 cu yd of concrete and

erecting gates and hoists (See Figs. 8 and 9). All electrical connections and the substation will be installed by specialized personnel. A contract for clearing the 30,000 acre reservoir was let in May, 1959, to the Wade Lahar Company, Mountain Home, Ark. This first project is to be finished and put into service in June, 1961.

The second project of the development (Logan Martin Dam) was started in July, 1960 with the first contract let for drilling and grouting a cutoff curtain in order to have some of this work well along before a general contract was let. A 640 ft section of curtain was completed in 6 months with 280,000 cu ft of grout (9 to 1, fly ash-cement) injected into cavities to depths of 250 ft beneath the surface. This was done by the split spacing method of drilling, with primary holes on 40-ft centers. Although this is the third dam down the river, it has the second greatest amount of flood control, and the license requires that the river projects be constructed in the order of flood control benefits. Foundation exploration in this vicinity has extended over approximately a 5-yr period in an effort to obtain the least troublesome location in the cavernous limestone area. Here the foundation problems are the most difficult of the entire river development.

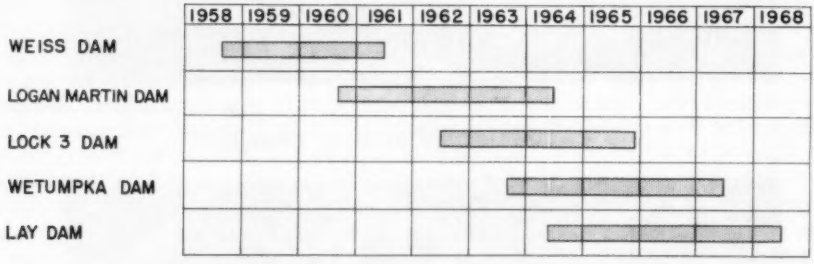

FIG. 7.—CONSTRUCTION SCHEDULE FOR COOSA RIVER HYDRO-ELECTRIC PROJECTS

A general contract for this dam was let in December, 1960 to the Guy H. James Construction Company of Oklahoma City, Okla. This job consists of moving approximately 2.25 million cu yd of earth and rock, pouring 175,000 cu yd of concrete, and completing the cut-off curtain by pumping an additional estimated 650,000 cu ft of grout solids.

Field Laboratories.—A rather complete soils laboratory had been established at the Weiss Dam project and has subsequently been moved to the Logan Martin project. In addition to the regular laboratory work in connection with soils and concrete at this project, an extensive soil investigation program is under way for future projects in this series. A second laboratory has been established for the investigation of suitable concrete aggregates for the Logan Martin Dam. This was done to establish a design mix and alternate aggregate sources prior to letting a construction contract.

Concrete.—The quantity of concrete is not large for each individual project and it would not be economical to construct aggregate cooling facilities for warm weather operation; however, a maximum of 80° F was specified for the

FIG. 8.—POWERHOUSE DIKES AT WEISS DAM

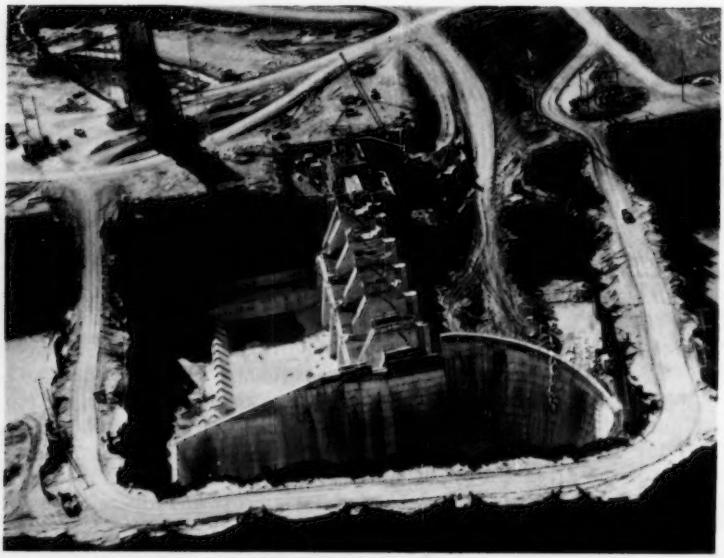

FIG. 9.—WEISS DAM SPILLWAY LOOKING EAST

temperature of concrete as it is poured into the forms. Construction schedules are considered so that the larger portion of concrete work will be done during the cooler months. Nighttime placing of concrete and hosing down the coarse aggregate have been used at Weiss Dam with success in the summer. Both fly-ash and 3 to 5% air-entrainment were used in the bulk concrete pours, fly-ash being secured from one of the company's steam plants.

PROJECT OPERATION PLAN

Power Generation.—Power operation will be effectively integrated with the company's existing hydro plants on the Coosa and Tallapoosa Rivers as well as with those plants proposed on the Warrior River. The installed capacity of existing and proposed hydro plants of the company on the Coosa River will aggregate approximately 680,000 kw.

Flood Control.—In the reservoir system a total of 850,000 acre-ft of storage capacity is provided for control of floods. Of the total, 75% is always useable, the remainder being available on a seasonal basis. The optimum use of this capacity, which is contained in Weiss and Logan Martin reservoirs, is obtained by the adoption of the so-called "induced surcharge storage" plan of operation. The overall project operations will be effectively coordinated with the existing Allatoona reservoir in Georgia constructed by the Corps of Engineers on one of the upper basin headwater tributaries. Construction of this series of dams will facilitate future developments on the Coosa River tributaries, and with such developments a still greater degree of flood control will be achieved.

In connection with routing floods through the reservoirs, as well as ordinary operation of the project, a comprehensive network of automatic radio stations for reporting precipitation or stream flow is to be installed. The data received will be coordinated with information from the U. S. Weather Bureau, Dept. of Commerce (USWB), thus securing most effective control of floods and use of flows for power generation. The Corps of Engineers and the company will jointly prepare an operations procedure that will be used for routing floods throughout the Coosa Valley. This is one of the requirements of the FPC license.

During flood periods generating equipment will operate at capacity. During periods of normal stream flow, reservoir releases will be made to satisfy load requirements. During low flow periods, withdrawals from power storage will be made, thus supplementing natural flows, meeting peak load requirements of the system, and making best use of available water consistent with applicable rules and regulations of the proper regulatory authorities.

BENEFITS

Power.—Of course the ultimate purpose of the development of the Coosa River by a private corporation is to produce power economically. This, it has been determined, can be done. The economics is justified by the furnishing of peaking capacity in a rapidly growing electric system.

Flood Control.—The substantial flood control benefits through the 600 mile length of the Coosa and Alabama River valleys resulting from project operations will approximate $500,000 per yr.

Industrial.—The development of this river will stablize the supply of industrial water and establish definite water levels desired by industry. The centers of existing industry on the river are located in Rome, Georgia, Gadsden, and Childersburg, Ala. The Birmingham industrial complex is only about 30 miles west of the midpoint of the river. Coal, gas, electric power, water, and labor are available in this valley, as well as highways, railroads, and a moderate climate, all of which are desirable factors for plant site selection.

Future Navigation.—The license requires that basic provisions be made at each dam for future construction of navigation facilities, and that the lakes be substantially continuous, thus providing a commercial waterway to Rome, Ga. In order to bring navigation from Mobile to the mouth of the Coosa River near Wetumpka, it will be necessary that three dams be constructed on the Alabama River. These are being studied by the Corps of Engineers and would further open up the industrial potential of the valley.

Recreational.—The completion of the Coosa River projects will create a total lake area of 118 sq miles with over 1500 miles of shore line. Several small impoundments have been built within the area of Weiss Reservoir. These have been stocked with fish so that when the new lake is formed, these fish will be well on the way to a "keeping" size.

CONCLUSIONS

The Coosa River may not be the first river to be wholly developed by private enterprise, but it is probably the largest development of an entire river by one corporation anywhere. These seven Coosa River hydro plants, together with other company hydro plants, 3 existing, and 3 proposed on the Warrior River (two under construction May, 1961), will create investor-owned hydro electric power production on the company's system of 1,200,000 kw.

ACKNOWLEDGMENTS

General project supervision is by Alabama Power Company's Engineering Department under the direction of E. R. Coulbourn, Vice President. Planning and detail designing is being done by Southern Services, Inc. under the direction of R. R. Randolph, Jr., F. ASCE, Manager, Hydro Project Section. Field investigations and project construction are under the direction of the Company's Construction Department headed by D. F. Elliott, Vice President.

To advise in the investigation, planning, design and construction of these projects, in October, 1956 a board of consulting engineers was established. The board consists of J. P. Growdon, F. ASCE, of Pittsburgh, Pa.; B. W. Steele, M. ASCE, of Miami, Fla.; George F. Sowers, F. ASCE, of Atlanta; S. H. Woodard, F. ASCE, of New York, N.Y. and J. M. Barry and H. J. Scholz retired company executives who advise on financing and system planning.

In the early stages of project planning, The J. G. White Engineering Corporation was retained, and since 1958 D. J. Bleifuss, F. ASCE, of San Francisco, Calif., has been retained as a special advisor.

The writer is indebted to P. L. Sharkey, Principal Hydrologist, Southern Services, Inc. and C. E. Gore, Jr., Geologist, Alabama Power Company among others.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

COOLING POND DESIGN IN THE SOUTHWEST[a]

By T. J. Cotter[1], M. ASCE and A. W. Lotz[2]

SYNOPSIS

The use of cooling ponds as a source of circulating water for central steam-electric stations is (in 1961) steadily increasing in the United States. The theoretical formulas by which heat dissipation rates may be computed have been established, but considerable engineering judgment must be applied in pond design to develop maximum cooling efficiency, because operating data are limited. Factors that should be taken into consideration in the design of a cooling pond are presented in addition to their application at two stations.

INTRODUCTION

Considerable stress has been placed on the water supply situation in the United States relative to the manner in which this natural resource might best be utilized for the maximum benefit of all. This is of particular interest in the Southwest in which forecasts of the nonconsumptive use—excluding hydroelectric plants, navigation and recreation—will exceed the estimated dependable water supply in 1980.[3] Furthermore, the estimated consumptive use will be about 50% of the dependable supply.

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

[a] Presented at the April, 1961 ASCE convention at Phoenix, Ariz.

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[2] Senior Mech. Engr., Cons. Engrg. Dept., Ebasco Services, Inc., New York, N. Y.

[3] "Availability of Water in the United States With Special Reference to Industrial Needs by 1980," by D. R. Woodward, Industrial College of the Armed Forces, 1957.

For electric utilities, water supply is a serious consideration in the development of plant sites. The growth of the utility industry and the attendent increase in plant size and circulating water requirements has resulted in more limited availability of plant sites where the supply is adequate for a one-through circulating water system. To meet this situation, there is a growing trend toward the use of artificial lakes and cooling towers that require a make-up water supply of less than 5% of the circulating water flow. In 1959, approximately 12% of the circulating water systems in thermal plants of investor-owned utilities was either a cooling tower or a pond as compared to 19% forecasted for 1980.[4]

It is evident, therefore, that future site selection and development may require consideration of either a reservoir or cooling towers as a means of cooling the condenser circulating water. The purpose of this paper is to present some of the salient design aspects of cooling ponds and to illustrate their application at two stations.

COOLING POND DESIGN ASPECTS

The design of a cooling pond is an art, because the many factors that affect the performance of the pond are so variable and peculiar to the individual situation that each design consideration must be evaluated separately on the basis of theoretical computations and past operating experience.

Several previous theoretical papers[5,6] have been written on the subject of heat dissipation from water surfaces and define formulas for the rate of heat loss. These formulas assume that the heated water discharged from a power plant is dispersed over the entire water surface. This assumption is valid under most operating conditions providing that the circulating water system is properly designed. Because each cooling pond has many features, particularly climatic, topographic and hydrologic that are peculiar to the individual pond, operating and research experience must be used to modify the conventional circulating water system design to effectively utilize the entire lake surface for heat dissipation. Furthermore, because there will be both horizontal and vertical temperature gradients within the pond, the design of the system should endeavor to obtain the coolest water which can be economically justified.

As a preface, to the analysis of cooling pond design aspects, a brief review of the effect of climatological conditions on the water temperature of natural lakes and cooling ponds would be of assistance in appreciating differences in cooling pond design.

A lake, on which no industrial heat load has been imposed, has a natural water temperature that is the result of a balance of heat input from sky and solar radiation and heat loss from back radiation, evaporation and conduction. When heat is imposed from an industrial source, the temperature of the water is artificially raised and increases the rate at which heat is lost until a new equilibrium temperature is attained. Because the natural heat input remains the same regardless of industrial heat input, the new equilibrium temperature is a function of the increased rate of radiation, evaporation, and conduction

4 "Water Resources Activities in the United States" by Select Committee on Natural Resources, U. S. Senate, Senate Resolution 48, 86th Congress.

5 "How to Predict Lake Cooling Action," by R. F. Throne, Power, Vol. 95, p. 86, September, 1951.

6 "The Use of Reservoirs and Lakes for the Dissipation of Heat," by G. Earl Harbeck, USGS Circular 282.

from the water body. These means of heat dissipation are dependent on air-water temperature differentials, wind velocity and vapor pressure differentials.

The effect of local climatic conditions on natural water temperatures is demonstrated by examination of atmospheric dry-bulb temperatures and natural water temperatures. The average annual dry-bulb temperatures across the United States are given in Fig. 1. Observe that the isotherms indicate an approximate range of 40° F to 70° F for the entire country. Particularly note that, within the State of Arizona, the full 40° F to 70° F range is experienced. The effect of local climatological conditions on natural water temperatures is illustrated[7] in Fig. 2 which shows isotherms of the approximate mean monthly temperature of water from surface sources for July and August. The range of summer water temperatures for the country is 60° F to 90° F. Natural water temperatures in Arizona average from 70° F to 90° F which represents practically the entire national range. The significance of these variations in natural water temperature in cooling pond design is that the maximum rise due to industrial heat load is limited to the difference between the maximum temperature of the industrialized pond and the natural water temperature. Therefore, if a maximum mean monthly water temperature in a cooling pond in Arizona is set at 100° F for the summer, then the location within the state would be extremely critical because the permissible rise due to industrial heat input would range from 10° F to 30° F.

In addition to dry-bulb temperature, the wind velocity has a substantial effect on the rate of heat dissipation and in particular, the rate of evaporation. Evaporated water leaving the lake surface increases the vapor content of the air immediately above the water surface. If this layer of air becomes saturated, then no further evaporation takes place. Reduction in the rate of evaporation would have a substantial effect on the total heat dissipation rate because the heat loss by the evaporation process in a cooling pond may average 60%, or more depending on local conditions. Furthermore, the rate of conduction also is reduced, thereby leaving only the heat loss by radiation unaffected. Air movement across the water surface causes turbulence and a scouring action that brings unsaturated air or air of lower vapor content in contact with the water surface. Fig. 3 shows some typical average annual wind movements all corrected to 25 ft above ground level for comparative purposes. The range in average annual wind movement is noted to be from 13 mph in the Texas panhandle to 3 mph along the Oregon coast.

The number of weather stations from which wind data are published for states in the Rocky and Appalachian Mountain ranges are too few to indicate the considerable range in local wind movement due to the configuration of mountains and valleys. Local airport wind data, in many instances due to the character of the terrain, show wind velocities that may average twice those recorded in valley locations. Thus, wind data that are applicable to the site under consideration are especially critical in the design of a cooling pond in the mountain states.

The vapor pressure of the air is a function of the water vapor content and the dry-bulb temperature. At constant temperature, an increase in water vapor content will increase the vapor pressure. As the dry-bulb temperature increases, the air can absorb more vapor. Thus, for a given quantity of water vapor, the vapor pressure will decrease with an increase in temperature. In

7 "Temperature of Water Available for Industrial Use in the United States," USGS Water Supply Paper 52.

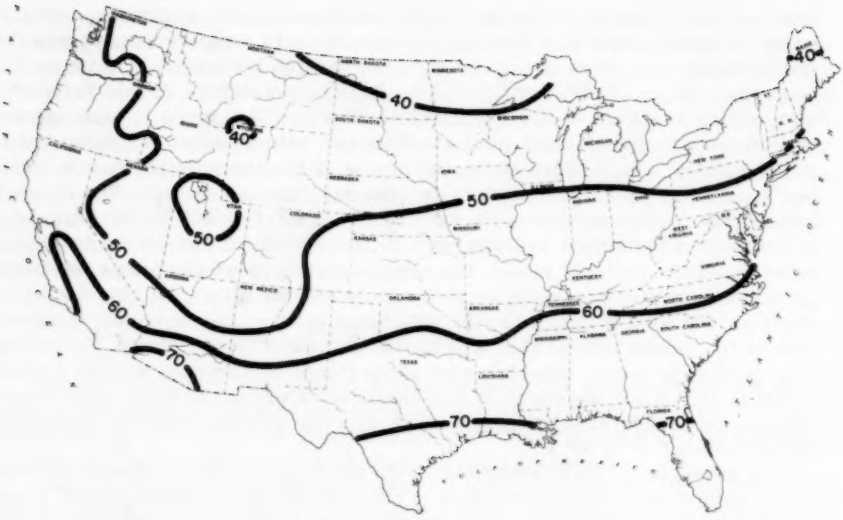

FIG. 1.—AVERAGE ANNUAL DRY-BULB TEMPERATURE

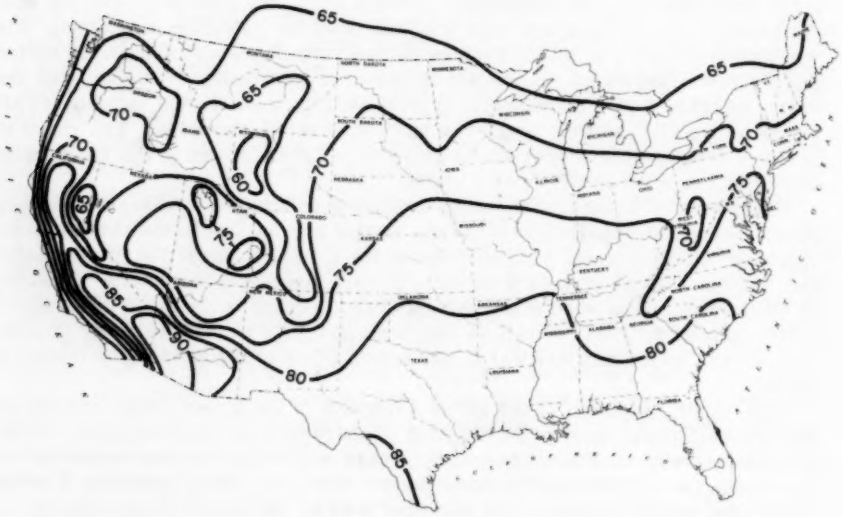

FIG. 2.—MEAN MONTHLY SURFACE WATER TEMPERATURE (JULY AND AUGUST)

FIG. 3.—AVERAGE ANNUAL WIND MOVEMENT, IN MILES PER HOUR

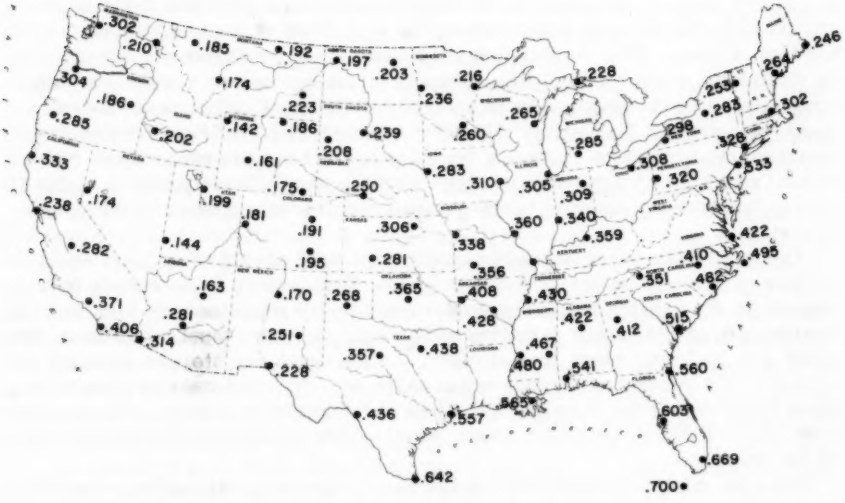

FIG. 4.—AVERAGE ANNUAL VAPOR PRESSURE (IN INCHES OF MERCURY)

a hot dry climate the vapor pressure will be low. The rate of evaporation, that is a function of the difference between the actual and the saturated vapor pressure, will be considerably more in a hot dry climate than in a cold humid climate. Fig. 4 shows some typical vapor pressures[8] at various weather stations throughout the country. The mean annual vapor pressure of 0.14-in. Hg at Yellowstone, Wyo. is shown as the lowest and the vapor pressure at Key West, Fla. of 0.70-in. Hg is the highest. In Wyoming, the climate is cold and dry whereas in Florida, the climate is hot and humid thereby resulting in these two extreme values. In Arizona the mean annual vapor pressure is known to range from over 0.3-in. Hg to less than 0.1-in. Hg.

These atmospheric conditions will affect the rate of heat dissipation from the cooling pond surface and will determine the base natural water temperature. It is left to the ingenuity of the engineer to utilize these natural processes to effectively dissipate the heat from the cooling pond surface and obtain the coolest water possible at the plant intake.

In addition to the atmospheric conditions, the engineer must take into consideration the geophysical features of the local terrain and the hydrology peculiar to each situation. Four major design features that should be taken into consideration in the pond design are pond depth, intake depth, separation of intake and discharge, and make-up water supply.

Pond Depth.—Acceptable depth of a cooling pond is dependent on (1) effect of dry-bulb temperatures, (2) rate of recirculation and (3) expectable drawdown during dry periods.

As previously stated, the natural water temperature is most clearly related to the dry-bulb temperature. Investigations have shown that the rate of change of intake water temperature with dry-bulb temperature is a function of depth. In shallow lakes, it is found that the water temperature down to a depth of about 10 ft to 12 ft will average within two or three degrees of the temperature at the top foot of water. This condition is the result of wind movement that normally is sufficient to overcome large thermal stratification and result in a rather small temperature gradient from surface to bottom. Furthermore, water to a depth of about 12 ft is materially affected by short term (three to five days) atmospheric conditions. A pond of less depth will result in even closer correlation (exclusive of industrial heating) with the dry-bulb temperature and will tend to increase the amount of surface area required for a given size of generating station.

The minimum water storage capacity of a pond should be at least equal to volume of water to be recirculated in 24 hr. This feature is necessary to take advantage of cooling during the cooler night hours when there is less natural heat input by sky and solar radiation and probably less heat input from the power plant due to lower plant load at night. As the depth and storage capacity increases, the percent of surface water taken into the condenser is proportionately smaller thereby reducing the effect of industrial heat input. To take full advantage of this condition the intake should draw water from the deepest part of the pond.

Once the minimum pond depth and storage is determined from the pond cooling performance viewpoint, study of the expected drawdown should be made based on analysis of stream flow records, pumped make-up, natural and forced evaporation, seepage, and other uses. From this information the normal and

8 "Evaporation From Lakes and Reservoirs," - Minnesota Resources Commission, June, 1942.

minimum operating level or depth of the pond can be established. Allowance should be made for future loss in storage capacity due to siltation if soil conditions within the drainage area so dictate.

Separation of Intake and Discharge.—The theory of pond surface cooling is based on the fact that the warm discharged circulating water on entering the pond will rise to the surface and spread out in a relatively thin layer. Theoretically, convection currents established by thermal differences should carry this warm water to all parts of the pond. The temperature of this surface layer would be in direct proportion to the distance from the point of discharge. The cooled water at the farthest point having a lower density will then drop down to a lower strata and establish a current counterflow to the surface current. On this basis, the logical location of the intake would be next to the point of discharge but at sufficient depth to prevent drawing some of the warm discharged circulating water into the intake.

Experimental and operating data from cooling ponds indicate, however, that some separation be provided between the point of intake and discharge due to the effect of wind movement on currents within the lake and the effect of currents induced by circulating water pumping. Published experimental data on the relation of wind velocity to water movement indicates that a wind of 5 mph will cause surface currents to move at a rate of about 750 ft per hr while a wind of 30 mph will cause a water movement of about 2,100 ft per hr. Below the water surface, the water moves along with the upper layers but at a velocity that decreases with depth. Test data on a cooling pond having an average depth of 8-ft and 7,500-ft separation of intake and discharge showed that when the wind direction is toward the intake, the warm water accumulates along the downwind shore and at the intake causing a marked increase in inlet circulating water temperature. Under the majority of wind conditions at this particular pond, return flow toward the intake would be counter to the wind-driven currents thereby resulting in good cooling performances.

Induced flow from the discharge to the intake might occur if separation is inadequate particularly for a large plant. Water temperature data, taken at a plant where the warm discharge water has the option of two paths, indicates that the warm water follows the path having the shortest distance between discharge and intake thereby demonstrating the effect of induced flow.

No precise distance can be established for the separation of the intake and discharge because the distance is a function of the quantity of heat rejected to the pond, wind direction, circulating water requirements, stream inflow, depth of pond, configuration of shore line, and location of the power plant. If the plant is located at one end of a long narrow pond, the warm discharge water may represent a relatively large proportion of the water volume in the vicinity of the plant discharge in which case the average temperature at that end of the pond may be several degrees higher than the average for the entire pond. This would be especially true in the case of a shallow lake of less than 10-ft depth. By discharging the water at a point near the opposite end of the lake, the over-all temperature of the water near the intake will be lower because it has had more time to cool due to the period required for recirculation. Construction of long canals may be uneconomical due to topography of the shore line, in which case a separation dike may be constructed parallel to the shore or jutting out into the lake proper. The latter course may be especially adaptable when the pond is relatively shallow.

Depth of Intake Structure.—In a natural lake, the surface waters are heated by sky and solar radiation. The action of the wind cools the upper layers and

causes the cooled water to descend. In addition, wind-driven currents as mentioned previously cause return currents underneath the surface layer. The combination of these two actions results in formation of three distinct strata namely a surface strata or epilimnion comprised of warm water at constant temperature, an intermediate strata or thermocline where there is a definite temperature gradient with depth and a deep strata or hypolimnion where the water is the coolest and at relatively constant temperatures. In a shallow lake (under 20 ft deep) the hypolimnion does not exist and the thermocline has only a small temperature gradient with depth. In a deep lake of 50 ft to 100 ft, vertical mixing may cause the thermocline to extend to a depth of 20 ft to 35 ft depending on wind velocity. During periods of calm, the thermocline may be of considerably less depth.

In a cooling pond, the principle of stratification remains the same except that the vertical temperature gradient in the thermocline is larger and may extend to a greater depth particularly in the vicinity of the intake due to induced flow from various layers. Cooling pond data indicate that on the average an intake structure taking in water at 25-ft depth result in inlet circulating water temperatures as much as two to three degrees lower than a structure that takes water at a depth of 10 ft. Normally this decrease in temperature would be sufficient to justify a deeper intake providing the topographic and geological conditions are not extreme. In some situations, it might be economical to build the intake below a 25-ft depth depending on the relationship of added cost for the intake structure as compared to the potential gain in improved station performance.

Inflow or Make-Up.—The source and quality of cooling pond inflow or pumped make-up that is needed to compensate for natural and forced evaporation and seepage losses, determine to a large extent the fluctuations of operating level of the pond and may have considerable influence on the maximum generating capacity that can be installed economically at a plant site. A stream-fed pond is usually designed for larger fluctuations in level than one receiving pumped make-up due to seasonal runoff and occasional droughts of long duration.

If make-up supply is to be pumped from wells, the level can be maintained more or less stationary. However, because ground water is often limited and expensive to develop it is rarely used for cooling pond make-up.

The amount of make-up required is determined from evaporation records in the area, estimated seepage loss and computed forced evaporation losses caused by heat input.

For an "off-stream" pond a study of the quantity and rate of water loss is essential to establish a pumping schedule and size of make-up water facilities. Such a schedule may be based on a rule curve to indicate a desirable rate of refilling after a period of drawdown. Proper allowance should be made for extreme conditions so as to avoid unbalance between available make-up and maximum drawdown and the resulting possibility of having to curtail electric generation to conserve water in storage until new inflow can be realized.

The two installations under construction (as of 1961) near Holbrook, Ariz. and Farmington, N. M. are illustrations of the application of cooling pond design principles and include some unusual features. They also demonstrate the desirability of obtaining climatological data that are recorded at or are representative of the pond site. At both of these sites, cooling towers had initially been considered for circulating water cooling but economics dictated the use

of cooling ponds. Water conservation was also a primary consideration at the Farmington site.

CHOLLA PLANT

The Cholla site lies in the valley adjacent to the north bank of the Little Colorado River 8 miles west of Holbrook. The predominant reason for selecting the Holbrook area was the availability of ground water of adequate quality and quantity from the Coconino Sandstone aquifer. The initial installation will be a 115,000-kw coal-burning unit. Coal will be mined from open pit mines at Gallup, N. M. and transported 120 miles by rail.

The topography and soil conditions at the site indicated a cooling pond might be constructed by using dikes to form three sides. Investigations showed that a 380-acre "off-stream" pond adequate to support the initial 115,000-kw unit plus an additional 175,000-kw unit could be constructed at a lower cost than cooling towers for the same capacity. The pond will have a storage capacity of 3,300 acre-ft and an average depth of 9 ft.

The top strata of soil at the Cholla site consists of sandy clay, thus insuring a relatively impervious reservoir and material suitable for compacted dike construction. The dikes with a maximum height of 14 ft will require 265,000 cu yd of fill. The natural soil is sufficiently impervious so that no core trench will be necessary and only stripping of top soil under the dike will be required. Dikes will be protected against wave wash by quarried Coconino Sandstone riprap. Freeboard of 3 ft will be provided on external dikes to allow for wave action and rise of pond surface during storm runoff that will discharge over an uncontrolled spillway.

The cooling pond design is unique because the pond is divided into two sections to form a "hot pond." The circulating water intake is located on the west end of the triangular shaped pond as shown in Fig. 5. A 2,300-ft long discharge canal will carry the warm circulating water northeasterly to the peak of the triangle. Initially, this separation of intake and discharge was considered adequate to provide reasonable distribution of the warm water in view of the shallow character of the pond. However, analysis of the wind rose indicated that practically all of the time the wind would blow from the southeast-southwest quadrant. Wind from this direction would cause the warm circulating water to build up along the north side of the pond and short circuit back to the intake without making beneficial use of the majority of the pond area for surface cooling. This situation has been indicated by temperature records at cooling ponds in the United States and confirmed by the use of a radioactive tracer (Iodine 131) in a cooling pond investigation at the Maitland Station of The Electricity Commission of New South Wales, Australia.

In order to remedy this situation, it is possible to either (1) route the discharge canal around the pond to the southeast corner or (2) extend a dike from the initial canal outlet across the pond in a southeasterly direction. Water temperature data collected at another off-stream pond utilizing a divider dike indicated that when the wind direction was counter to the path of flow through the discharge section of the pond, a high rate of cooling was experienced. This condition was attributed to the fact that the water within the pond discharge section had a high temperature level over a considerable area thereby maintaining a large temperature differential between the ambient temperature and

the water temperature. The net effect was an increase in the rate of heat dissipation and over-all pond cooling efficiency. Without a dividing dike to constrain the movement of the warm discharge water, the warm water would spread over the water surface and cool thereby reducing the magnitude of the water-air temperature differential and correspondingly the rate of heat dissipation.

At Cholla, this so-called "hot pond" concept will be further improved on by the construction of an inverted weir at the outlet of the "hot pond" as shown in Fig. 6. This weir will permit only the coolest water to leave the "hot pond" because of density differentials. Without the inverted weir, counter flow of cool water at lower levels into the "hot pond" would result from convection currents. A design velocity of one fps under the weir is used based on investigations conducted by the State Electricity Commission of Victoria, Australia. This veloc-

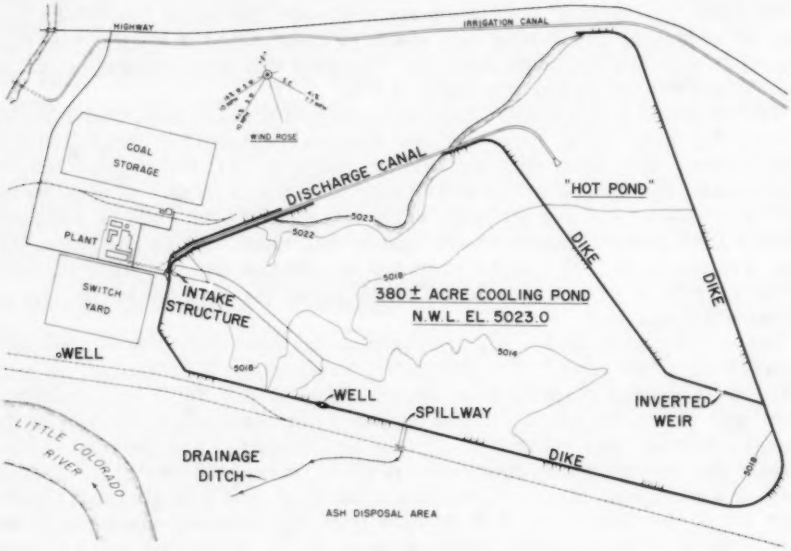

FIG. 5.—CHOLLA SITE DEVELOPMENT PLAN

ity will be sufficient to prevent the counter flow of cool water and also minimize the drawing of warm surface water down under the weir.

An unusual feature is that make-up water to the pond is obtained from wells. Usually where a surface water supply is unavailable, the higher water loss from a cooling pond due to natural evaporation and forced evaporation as compared to forced evaporation from a cooling tower results in substantial penalty against the cooling pond for greater make-up water requirements. However at Cholla, atmospheric conditions are such that the evaporation rate from the cooling tower is quite high as compared to the additional evaporation, or forced evaporation, from the cooling pond. The annual average evaporation loss, natural plus forced losses, from the cooling pond after installation of two units is practically equal to the evaporative and driftage losses from cooling towers.

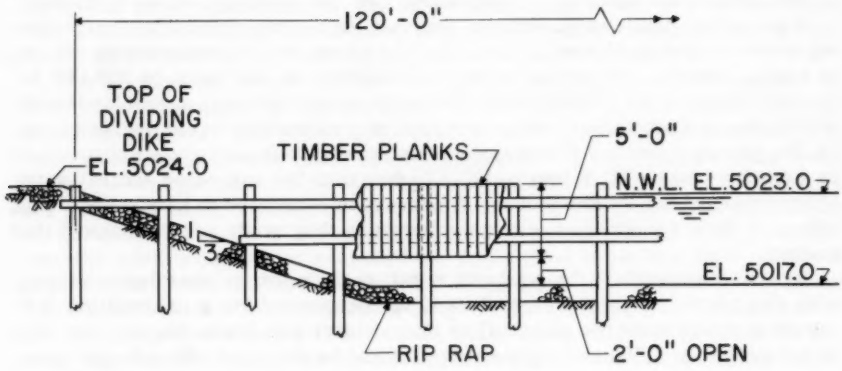

FIG. 6.—INVERTED WEIR

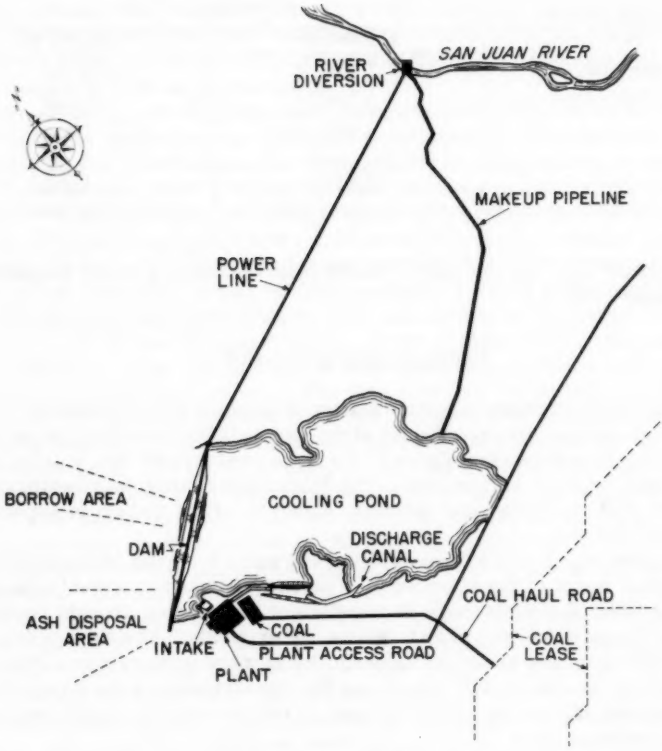

FIG. 7.—FOUR CORNERS DEVELOPMENT PLAN

The estimated make-up requirements for the pond are shown in Table 1.

The cooling pond design criteria was that the average annual inlet circulating water temperature would be equal to the temperature obtained with the use of cooling towers. Assuming a total installation on the pond of 290,000 kw (1-115,000-kw and 1-175,000-kw unit) operating at an average annual load factor of 85%, the estimated heat load over the entire pond surface would average about 72 Btu per sq ft per hr. The average annual inlet circulating water temperature is estimated to be 72° F that is 17° F higher than the estimated natural water temperature of 55° F. The "hot pond" concept is estimated to have a beneficial effect of 3° F cooler inlet water temperature than would prevail without that feature.

The shallow depth of the pond will result in the water temperatures varying with short term ambient temperatures. To compensate for this condition during the summer when the plant will be operating at high loads, the surface area of 1.3 acres per 1,000 kw is greater than would be required with a deeper pond.

TABLE 1

Average Annual	Acre-feet per Year	
	115,000 kw	290,000 kw
Forced Evaporation	810	1,700
Net Natural Evaporation	1,540	1,540
Blowdown	585	810
Seepage	80	80
	3,015	4,130

It is estimated that the maximum 24-hr inlet circulating water temperature will not exceed 100° F.

FOUR CORNERS PLANT

The Four Corners Steam-Electric Station is located in the northwest corner of New Mexico on the mountain desert of the Navajo Indian Reservation as shown in Fig. 7. The site is located adjacent to a large coal deposit that will be mined for plant use by open pit methods. The initial installation will comprise two 175,000-kw units whereas the ultimate capacity of the station is expected to exceed 1,000,000 kw.

The San Juan River which derives 95% of its water from the Southern Rockies flows 3 miles north of the plant site. Whereas the average annual flow of the San Juan is about 2,600 cfs there is insufficient flow during drought periods to support existing municipal and irrigation water rights. The Navajo Dam will partially alleviate this situation although the storage in Navajo reservoir will be insufficient to insure full water use for the irrigation season in prolonged drought periods. Groundwater in this area is not adequate in quality or quanity for central station use.

A water permit has been issued by the state of New Mexico to divert into a reservoir 33,000 acre-ft per yr of unappropriated water from the San Juan

River for power plant usage. The appropriation is firm on an annual basis therefore some storage requirement is necessary when pumping must be curtailed due to insufficient flow to supply need of prior rights.

The preliminary plans for the development of this site initially called for the construction of an off-stream reservoir into which water from the San Juan would be pumped. The reservoir would serve as a source of water supply to the power plant for boiler make-up, equipment cooling water, and cooling tower make-up. Comparison of a circulating water system comprised of cooling towers receiving make-up from an off-stream reservoir versus a system utilizing an off-stream reservoir for a cooling pond indicated that the total water use from the cooling tower plan exceeded the water used by cooling pond-reservoir plan. The high rate of water use by the cooling tower plan resulted in a reservoir for the ultimate plant almost as large as the cooling pond. Comparison of the water losses showed that the natural evaporation loss from the two reservoirs was about equal. The loss from the cooling towers due to evaporation and driftage was substantially higher than the evaporation loss from the cooling pond due to discharge of the warm circulating water. Thus, for the same quantity of make-up, more generating capacity could be installed at a lower investment with a cooling pond development than with the cooling tower plan.

The cooling pond will be located 3 miles south of the San Juan River in a bowl-shaped wash on the mesa 300 ft above river level as shown in Fig. 8. The pond will have an area of 1,275 acres, a capacity of 39,000 acre-ft and an average depth of 29 ft as shown in Fig. 9.

The necessity of obtaining meteorological data either at or representative of the cooling pond site was particularly stressed at Four Corners as related to make-up water requirements. Preliminary studies relating to the feasibility of the project used evaporation data collected in the Farmington area for over 40 yr. These data were collected from evaporation pans of different types and at different locations. The first 30 yr of data were collected from a floating pan set in a swampy area between the San Juan and Animas rivers. The next set of data were collected from a standard Class A Weather Bureau pan set in the flood plain of these rivers. The last data were collected from a Class A pan located in the irrigated valley of the Animas River. The cooling pond site, however, is on a high desert mesa which is exposed to prevailing winds from the southwest and quite dry. The data from the Farmington pans, although taken only 20 miles away, were not a true indication of evaporation rate at the pond site because the evaporation loss probably was affected by the surrounding marsh lands and by wind velocity and direction determined by the local terrain. Therefore, evaporation estimates were made by extrapolating Farmington data based on short term records at Shiprock, N. M. where the pan location was more representative of the pond site conditions. The average rate of evaporation at the plant site was estimated to average 37% higher than at Farmington and naturally had a material effect on make-up requirements.

Comparison of the Shiprock and Farmington climatological data also indicated that the prevailing wind direction at Shiprock is southwest whereas at Farmington the direction is easterly. This change in direction at Farmington demonstrates the protection that the gorge enjoys from the mesa winds and the channelling effect of the gorge. Likewise, this change in direction substantially affected consideration of plant orientation, discharge canal and intake-skimmer wall designs.

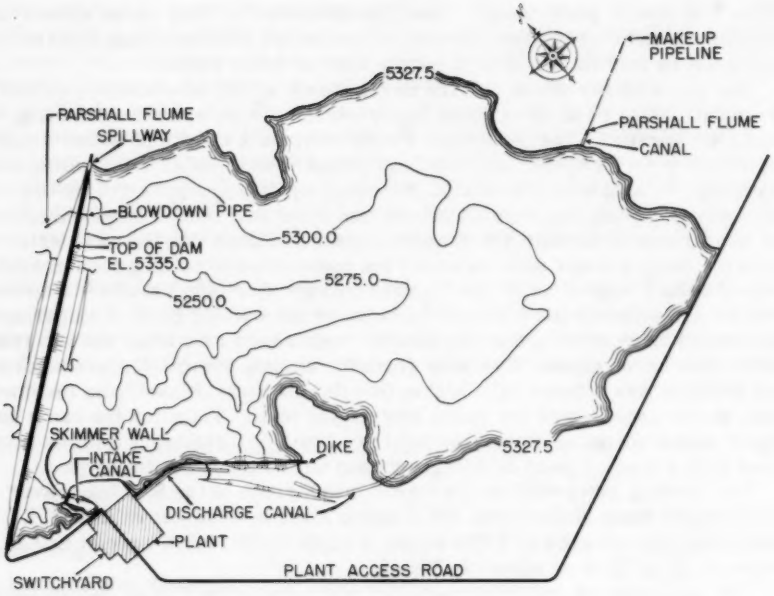

FIG. 8.—FOUR CORNERS COOLING POND

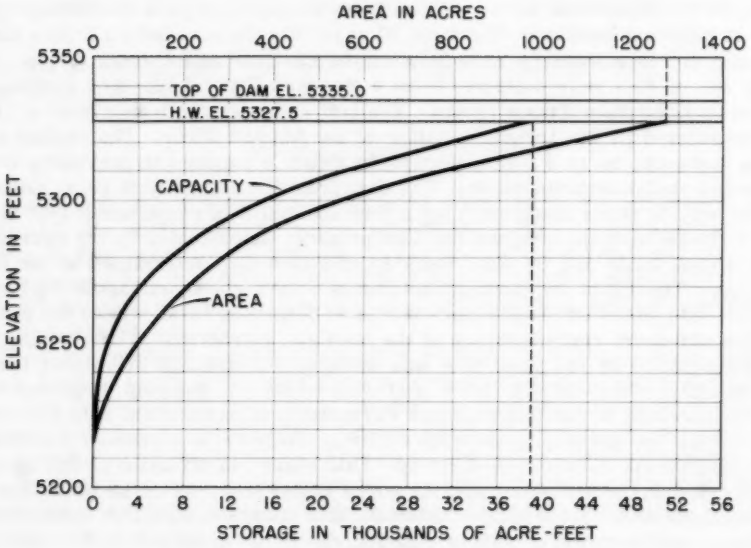

FIG. 9.—AREA-CAPACITY CURVES

The amount of generating capacity that can be installed at the Four Corners plant is directly proportional to the quantity of make-up water available from the San Juan River. The cooling pond will have a full pond area of 1,275 acres. If the pond could remain substantially full, approximately 1,500,000 kw could be supported by the pond. However, after allowance for curtailment of make-up pumping during a drought period, the cooling pond in the summer might be drawn down 7.5 ft that would result in a minimum pond area of 1,080 acres. This area would be sufficient for a 1,270,000-kw installation on the premise that the maximum 24-hr inlet circulating water temperature will not exceed 100° F. The surface area requirement per unit of installed capacity is about 0.8 acres per kw at Four Corners as compared to 1.3 acres per kw at Cholla. This difference is primarily due to the "flywheel" effect that will result from the greater volume and depth of water in storage at Four Corners pond. The average water temperature will be less susceptible to short term variation in atmospheric conditions and plant operation than would be expected with a shallow pond.

A common situation at both Cholla and Four Corners, is the problem of increase in dissolved solids concentration of the pond water resulting from practically no inflow except pumped make-up and little, if any spill, from the pond. Based on good cooling tower practice, the dissolved solids concentration should be limited to about 1,400 ppm which is the upper limit that can be properly treated to prevent excessive scale formation on condenser and heat exchanger tube surfaces. At both plants the rate of build-up of excessive concentrations is minimized by the use of water from the pond for hydraulically sluicing ashes into disposal areas. This means of "blowdown" is estimated to keep the dissolved solids concentrations in the pond water within manageable levels.

The estimated make-up water requirements for the Four Corners pond are as follows for a 1,270,000-kw plant:

Average Annual	Acre-feet per Year
Forced Evaporation	9,600
Net Natural Evaporation	6,800
Blowdown (5,500-2,500 seepage)	3,000
Seepage	2,500
Other Plant Usage	1,100
Total	23,000

Under drought conditions and depending on the method of development of the Four Corners plant, under which 1,500,000-kw might be installed if the cooling pond is kept full, the annual make-up water requirements are estimated to increase to over 30,000 acre-ft.

Cooling Pond Dam.—Geologic formations present in the general site area are Picture Cliff Sandstone of near shore marine origin and Lewis Shale of marine origin. There is no sharp line separating the sandstone from the underlying shale but rather a gradation zone of siltstone and sandy shale. The dam site and western half of the reservoir are in this gradation zone.

The siltstone and shale at the dam site show generally two zones of varying degrees of weathering. An upper zone 5 ft to 10 ft thick of highly weathered intercalated siltstone and sandstone along with gypsum stringers. This is underlain by a zone 10 ft to 30 ft thick of partly weathered siltstone and shale containing gypsum filled fissures.

The 6,000,000-cu yd earth fill dam will be 6,800 ft long with a 133-ft maximum height. A typical section of the dam is shown in Fig. 10. The highly weathered zone of siltstone and sandstone will be stripped from the base and a 20-ft to 40-ft cutoff trench will be excavated through the partially weathered zone to competent rock. The cutoff trench backfill and impervious core consists of compacted loess borrowed from the bottom of the cooling pond wash. The shell zones consist of sand and gravel fill from a high river terrace 3 miles from the dam site.

River Diversion.—The San Juan River has a flow range of 400 cfs to 100,000 cfs, and a silt load that averages 3,300 ppm and sometimes exceeds 60,000 ppm. The river slope averages 10 ft per mile and in alluvial fill areas has a tendency to meander. The river diversion (shown in Fig. 11) will be located at the base of a 200-ft bluff on an outside bend of the river to insure a stable channel and to obtain scouring velocities on the pumping side of the river.

The silt problem coupled with the wide stream flow range required the design of diversion facilities that would permit the maximum permissible pumping time without incurring excessive maintenance.

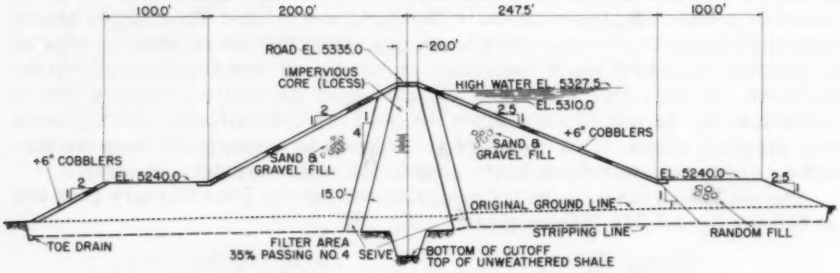

FIG. 10.—TYPICAL DAM SECTION

Minimum pumping level will be maintained by a sheet pile weir immediately downstream of the intake. The south or intake abutment of the weir consists of a sluiceway mounting a 6 ft-by-20-ft flat gate. The high velocity flow through the sluiceway will prevent accumulation of sand and gravel in front of intake.

The intake structure (Fig. 12) will mount coarse trash racks, closure gate, traveling bar rake, sand traps, and make-up pumps. Sand entering the pump chamber will be deposited in a concrete hopper and will be conveyed away through a 5-in. suction hose connected to dredge type sand pumps each designed to handle 10% solids at a maximum rate of 9 cu ft per min. This system is estimated to be capable of removing 75% of the suspended solids when the river water has a load of 10,000 ppm.

Analysis of river flows and silt loadings of the San Juan River indicated that under the worst conditions of low flows and high suspended matter, make-up water pumping to the cooling pond would be restricted to 250 days annually. Total pump capacity of 30,000 gpm was selected as being capable of providing the estimated 33,000-acre-ft make-up required annually. These same pumps will be used for the initial filling of the pond in approximately a year and one half of pumping based on 400 days of actual operation. The make-up pumps

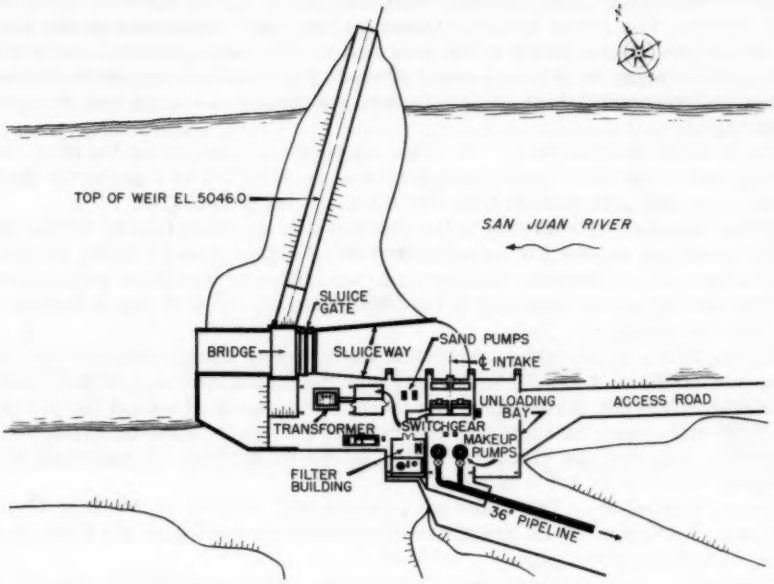

FIG. 11.—RIVER DIVERSION PLAN

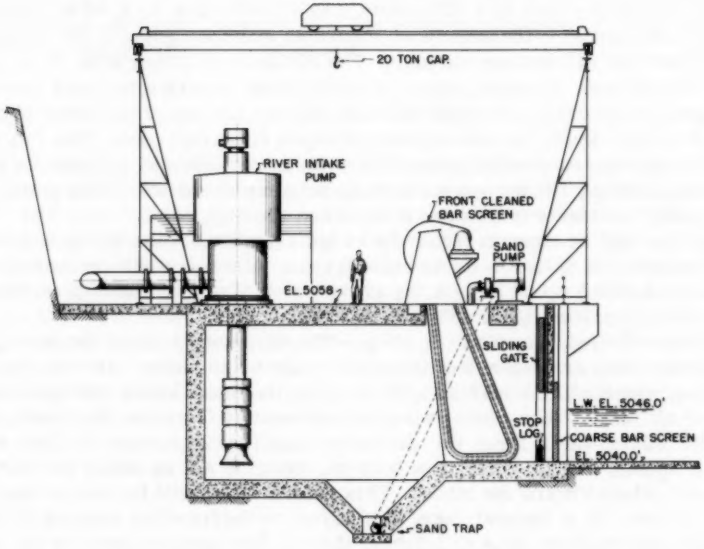

FIG. 12.—RIVER DIVERSION INTAKE

will be sized at 15,000 gpm each at 450-ft head and driven by 2,000-hp synchronous motors. The pumps are four stage, turbine type, unbalanced design having closed shaft with clear water lubrication. The impellers and bowls will be neoprene coated to reduce excessive wear. The hydraulically operated pump discharge valves will have stellited edged stainless steel discs and neoprene coated body.

Clear water lubrication for the make-up pump and sand pump bearings and cooling water for the motor bearing oil will be provided by a pressure filter system that will take suction from the make-up pump discharge.

Water hammer and surging in the pipeline will be controlled by setting the pump discharge valves for an estimated 90-sec opening and closing periods. The valves will be operated through a nitrogen filled accumulator pressurized to 1,000 pse to prevent draining of the pipeline in the event of power failure to the river diversion.

During filling of the make-up pipeline the make-up pumps would exceed the allowable NPSH and result in cavitation if the discharge valve is wide open. Similarly, if the discharge valve was throttled to maintain head of the pumps, the valve disc would be subject to cavitation. Therefore, an 8-in. bypass will be installed around the discharge valve by which the velocity head loss will dissipate as much as 350 ft of head.

Make-Up Pipeline.—The make-up pipeline will consist of 360 ft of 36-in. steel pipe, 6,800 ft of 36-in. prestressed concrete cylinder pipe and 6,700 ft of 42-in. concrete culvert pipe.

The steel pipe of penstock type construction will be used for the rise out of the river gorge. The 200-ft bluff consists of weathered sandstone and shale containing large gypsum filled fissures. Although the river face of the bluff is vertical, there is evidence that the bluff is continuously sluffing. A portion of the rock will be cut back to a 45% grade, and a continuous 10-ft wide concrete slab will be anchored to the hillside to insure an adequate support for the pipe. The pipe saddles will rest on the slab. The cut area on either side of the slab will be covered with gunite to prevent moisture from entering the rock fissure.

The prestressed concrete pipe will extend from the top of the river bluff to the top of a ridge, which is about midway between river and pond. The line will contain two air-vacuum relief valves located at major changes in grade to prevent surging during filling, pump start-up or pump shutdown. This portion of the lines will terminate in a surge chamber at the high point.

The water will be conveyed from the surge chamber to the pond in a gravity line constructed of 42-in. concrete culvert pipes. Manholes will be located approximately 1,000 ft apart to vent the gravity line. The pipeline will terminate at a Parshall flume equipped with recorder totalizer.

Condenser Circulating Water System.—The circulating water intake will be located at the west or deep end of the pond on the south shore. The discharged circulating water will be carried 5,000 ft along the south shore through canals and diked off inlets. The winds, which are primarily from the southwest, will carry the warm water from the discharge canal outlet across the lake away from the intake. The counter flow current, which is set up below the surface strata will return toward the intake. This return water will be cooler than the surface strata. In a natural lake, this layer or thermocline extends 20 ft to 35 ft below the surface. It is anticipated that the heat load imposed on the pond will depress the depth to which the thermocline exists. An arbitrary depth of

45 ft has been assumed. If the intake takes suction from this depth or deeper then the coolest water available will be obtained.

On this premise a deep intake was considered but less costly approach was developed. The intake will be located at the head end of an arroyo with a 14-ft depth at low water. A skimmer wall will be placed across the entrance of the arroyo. The wall of steel "A" frame and precast concrete slab construction,

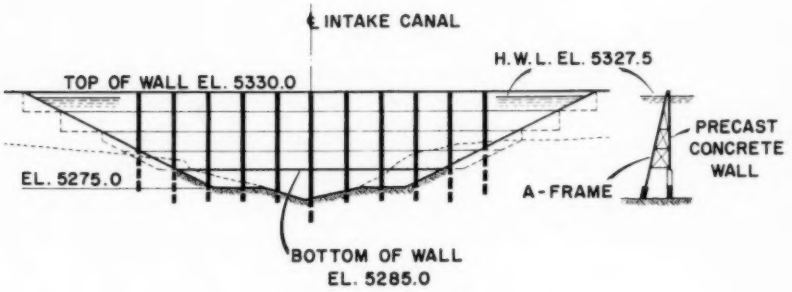

FIG. 13.—INTAKE CANAL SKIMMER WALL

as shown in Fig. 13, will extend 40 ft below low water and is designed to permit passage of water under the wall at a velocity of about 1.25 fps when ultimate generating capacity is installed.

CONCLUSIONS

Although numerous formulas and heat balances have been published for cooling pond design these can only be considered a guide for computation of pond size and expected temperature rise. Judgment factors based on past experience with ponds of various physical characteristics under different climatical conditions are of the greatest importance when evaluating the design that will result in the most efficacious use of the pond surface for heat dissipation and put to the most beneficial use the water available. Several physical features have been described; however, the cooling pond has to be built according to (1) the terrain as exemplified by the shallow Cholla pond constructed on flat ground and the deep Four Corners pond with 40,000 acre-ft of storage, (2) the climatic conditions, and (3) the available make-up water supply.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. PO 2, July, 1961.

DESIGN OF SELF-SUPPORTED STEEL TRANSMISSION TOWERS[a]

Closure by R. N. Bergstrom, J. R. Arena, and J. M. Kramer

R. N. BERGSTROM,[18] F. ASCE, J. R. ARENA,[19] F. ASCE, and J. M. KRAMER.[20]—It appears that all of the discussers have agreed on one important point. This point is that the present N.E.S.C. does not present adequate structural provisions to supply a proper criteria for tower design throughout the United States. They also underscore the related fact that tower weights can vary according to the designers interpretation of the code and to the freedom of choice afforded the designer by the many omissions in the code.

The writers agree with Hollick that the ultimate compression formulas as plotted in Fig. 9 as "allowable unit stress" should be called ultimate unit stress. Hollick's questioning of the use of decimals to the second place for the overload factors as being inconsistent with some of the assumptions made in tower design is well taken, but the overload factors shown are exactly as listed in the N.E.S.C. This merely illustrates one of the many illogicalities of the existing system.

Dugan presents a fine analysis showing the result of great effort and preparation, and he did especially well in presenting a competing tower design. The writers do not quite understand why Dugan was skeptical about the economical aspect of the rectangular tower. A closer look at the weights of the two designs will show that the design described by the writers was lighter than the square tower design presented by Dugan. The original paper did emphasize that the 12,306 lb tower weight included the weight of the steel grillages. Without the grillages, this tower weight 10,300 lb and it is almost 1,000 lb lighter than Dugan's weight of 11,200 lb without grillages. Dugan's tower is 2 ft 6 in. shorter and the spread of the base is 21 ft as compared to 25 ft for the original tower. This, of course, means greater foundation loads for the 21 ft tower base, and presumably they would be more expensive. The original tower also satisfied a design condition not mentioned in the paper that a 14 ft by 8 ft area of access must be provided through the tower for passage of farm equipment.

With regard to compression formulas, the writer notes that the cited straight line formula for A7 steel practically coincides in the L/r range of over 70 with the modified secant formula of the Priest and Gilligan text. The writers believe that in the L/r range below 70, the secant formula is unduly conservative. There is a wealth of test data to substantiate this fact.

The comparison of the economy of high strength steel to A7 steel is based on a straight line formula for A7 and the secant formula for high strength steel.

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[a] June 1960, by R. N. Bergstrom, J. R. Arena, and J. M. Kramer (Proc. Paper 2525).

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Because these formulas were selected for the tower design, the comparison was made on this basis.

The writers differ strongly with Dugan's reluctance to place a limit on L/r for tension members and believe emphatically that there is a great need to adopt a general requirement that would assign a maximum L/r to tension members. The writers do not agree that where a tension member serves as a secondary brace, it should always be governed by a maximum L/r of 250 as other considerations also affect it.

Regarding Dugan's comments on using the ultimate bolt strength, as compared to the use of the yield strength for tower members, this procedure follows current practices in tower design and the writers have never heard of a bolt failure during a tower test.

The writers are encouraged in the unanimity of the discussers in sharing the hope that this paper and the discussions resulting therefrom will serve as an impetus for improving existing codes and establishing uniform design criteria.

DIGITAL COMPUTERS FOR TRAIL-LOAD ANALYSIS OF ARCH DAMS[a]

Discussion by G. S. Sarkaria and R. P. Wengler

G. S. SARKARIA,[9] M. ASCE.—An electronic computer program, or a series of programs, that would relieve the engineer of the more repetitious, and often rather monotonous computations that comprise the preliminaries of a trial-load analysis is indeed a welcome contribution to arch dam design. When such a program is prepared by an engineer with many years of experience in the stress analysis of arch dams, it is bound to be thorough and capable of producing as satisfactory results as the "manual" method.

With the burden of the prosaic computations off his mind, the designer can now explore theories and concepts that were previously considered far out of the accepted line. Many of these ideas did not progress beyond the hypothetical stage because of lack of time and funds required to investigate their practicability. Others have awaited the improvement of quality of building materials and methods as well as design techniques to prove their usefullness.

Some of these trends and concepts are examined herein, with the hope that they would be investigated with the aid of the electronic computer programs presented by the author.

The most commonly accepted approach to arch dam design is to start with a conservatively thick structure, and then to trim it as indicated by detailed stress analysis. This engenders in the designer the gratifying feeling that he has saved so much concrete, and that the economy achieved justifies the cost and refinement of the stress study. If, on the other hand, the start is made on a minimum structure, which according to experience would have higher than permissible stresses, and then it is thickened where indicated by analyses, the designer would still be motivated by the urge to economize. However, the resultant structure will be thinner than the one designed by the step-by-step trimming procedure. With the aid of the electronic computer programs, it should be possible to establish the initial minimum designs for different heights and types of dam sites.

Curvature in the vertical plane of an arch dam, and consequent overhang at the upstream and downstream faces, is another feature that should be studied for its influence on the stability of the dam. While many dams with pronounced double curvature or cupola shapes have been built abroad, most arch dams in the United States are without this shape characteristic. The effects of curvature in the vertical plane on load distribution, deflections, and stresses in the dam should be evaluated by a series of analyses. No particular complications are encountered in using the trial-load method for double curvature dams. Therefore, the fastest and the less expensive method that could give a quanti-

[a] August 1960, by L. R. Scrivner (Proc. Paper 2568).

[9] Superv. Engr., Internatl. Engrg. Co., Inc., San Francisco, Calif.

tative idea of the influence of this variable should be welcome. The results of such an investigation could also be correlated to a study of increase in formwork costs due to two-dimensional curvature and overhang.

Vertical normal stresses or principal stresses almost normal to the foundation, acting at the upstream face of an arch dam near its base tend to be tensile. In some designs, it is possible to reduce or eliminate such tension by thinning the cantilever section in such a manner that more of the external load is transferred to the arches. However, if the stresses in the arches are already approaching the permissible limits, it would be necessary to thicken the arches, that is, the dam as a whole. An alternative would be to induce vertical compressive forces in the dam at the upstream face by means of post-tensioning cables anchored to the foundation. Where the foundation bedrock is suitable, this method has been successfully applied to gravity dams and retaining walls. The degree of post-tensioning that would be required could be made part of the normal trial-load analysis with the aid of the digital computers.

Another contribution from an extensive electronic computer program, such as the one described by Scrivner would be to develop design data tables and curves which would be useful to arch dam designers who may not have access to electronic computers. The following are some of the basic data the writer has in mind:

1. Deflections of variable-thickness arches at quarter points due to unit uniform and triangular radial, tangential, and twisting moment loads.
2. Moments, thrusts and shears in variable-thickness arches at quarter points due to unit uniform and triangular, radial, tangential, and twisting moment loads.

Similar curves, but of much limited applicability, have been published elsewhere.[10] If such data are published, it would offer an inducement to more designers to delve into the trail-load method, and would further stimulate interest in design of arch dams. It is realized that the task of obtaining such data, considering all the variables involved, would be monumental. The writer, however, has no doubt that with the electronic digital computer programs at their disposal, the author and his colleagues will be able to accomplish it in the near future.

R. P. WENGLER,[11] A. M. ASCE.—Schrivner's paper has great significance in the field of arch-dam design because as more and more portions of the analysis become programmed for the computer, one severe criticism of trial-load can be eliminated. This criticism is that the analysis when made on desk computers requires considerable time and many man-hours of work. The writer's experience generally agrees with the author in that in certain repetitive parts of the problem, the computer shows cost advantages as high as 15 to 1. It is certain that the reduction in cost of trial load analyses will bring about a re-evaluation of the relative merits of trial load versus model tests in arch dam design.

The paper is interesting in accomplished programming and also in future programs. In the programs already written, the writer notes that the shape of the unit arch loads were taken exactly as in the traditional trial load method. The triangular arch loads can begin only at quarter points. This enabled the

10 "Arch Dams with Arches of Variable Thickness," by A. L. Parme, Structural Bur., Portland Cement Assn.

11 Engr., Harza Engrg. Co., Chicago, Ill.

author to set up a card library of f(X) values from existing USBR tables to be used as input to his program #800. The obvious advantage to this scheme is that considerable saving of machine time can be made by this use of existing data. It appears to the writer, however, that by using this technique, the author is forced to make an assumption of uniform arch thickness between quarter points. If this is true, does the author have any experience on the effect of this assumption on the accuracy of the computations? It is noted in Fig. 4 that the author apparently intends to write a new program where the assumption of uniform thickness can be made over smaller central angles and will no longer be limited to the quarter points. Does the accuracy of the computation require this change or is this only slight improvement? This change will, as emphasized by the author, require the computation of f(X) by the computer. It has been the writer's experience that the f(X) computation requires a significant portion of the overall program. With a program written by the writer, the time required to solve for 13 unit radial nonsymmetrical radial loads is approximately 1 hr per arch as compared to 1/2 hr by the author. While the writer used a slightly slower computer and although there are other variables involved, the writer feels that a considerable part of the difference can be explained by the f(X) computation. The equation in the writer's mind then arises; is the increased accuracy worth this added time?

It is noted that the author intends to compute f(X) from the power series of X in radians. This idea certainly appears to have an advantage as the writer experienced considerable difficulty in programming the integrals for the load types shown in Fig. 4. The difficulty arose in trying to take advantage of loops in order to shorten the program sufficiently to be handled in the storage space available. It appears to the writer that possibly the author intends to avoid the difficulty by using the power series. In any case, it would be interesting to know exactly what the author's experience has been on the advantages of programming the power series versus the intergrals.

Apparently the author has not as yet made up his mind on how to solve for the deflection adjustments. He mentions two methods such as an iteration process and a solution of a large group of simultaneous equations. It is not clear what he means by iteration, but possibly it could be that the designer would decide on a trial load distribution and the computer would check it. It could also mean that the computer would solve the simultaneous equations by an iteration procedure. In any case, the writer's preference has been to set up deflections equations and invert the square matrix on the left hand side which is dependent only on the geometry of the dam and its foundation constants. This inverted matrix is then multiplied by the right hand column matrix, which contains the load variables, giving the final solution. A large percentage of the time required for solution is taken up by inverting the left hand side. However, for any given geometry, this inversion need only be conducted once. The multiplication by the right hand side for any number of different loading cases can then be conducted quite rapidly. It is of advantage only when two or more loading cases are to be investigated.

DESIGN OF ARCH DAMS BY TRIAL-LOAD METHOD OF ANALYSIS[a]

Discussion by G. S. Sarkaria and J. A. Veltrop

G. S. SARKARIA,[8] M. ASCE.—By showing results of different steps or types of analyses that comprise the evolutionary process for design of an arch dam, Copen has correctly presented the trial-load method as a logical and systematic procedure. All analytical methods of determining stresses in an arch dam, particularly the trial-load method, are often subjected to criticism because of the length of time required to accomplish the results. Sometimes the need for accuracy of detail, as required for a complete trial-load analysis, is questioned. Validity of the overall theory of the trial-load method is generally not disputed. Copen has indirectly answered many of these objections. Further examination of some of the assumptions and procedures should, in the writer's opinion, be helpful in explaining the mechanics, scope, as well as limitations, of the method.

Abutment and Foundation Deformation.—The arch and cantilever elements are assumed to abut against or rest on elastic abutments or foundation. Foundation deformations are estimated due to loads and moments acting normal to and in plane of a finite foundation area. These are based on an average sustained modulus of elasticity of the foundation rock in compression or tension. Where rock characteristics show great variation in elastic properties, different values for the modulus for different portions of the foundation can be used in the analysis.

The assumption of elastic fixity at the abutments and foundation, which considers elastic deformation and rotation of the foundation and the dam, should not be confused with the assumption of hinged or semifixed abutments. The latter presumes body rotation of the dam relative to the foundation, which can occur only if the bond between concrete and the foundation is substantially broken or when a slide or hinged joint is purposely provided. Copen's comments, and data from USBR designs, regarding the influence of elastic properties of the foundation on deflections and stresses in arch dams would be a useful addition to this paper.

Confirmation of Trial-Load Analysis Results.—Although considerable data are now available for checking the results of analytical computations against observations made on models and prototype structures, complete confirmation between the two cannot be claimed. Radial, tangential, and angular deflections obtained from complete trial-load analyses are generally in good agreement with those observed on completed dams. However, this is not true of stresses, which when computed from strains observed in models and full-size structures often vary greatly from analytical results in magnitude as well as distribution.

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[a] August 1960, by Merlin C. Copen (Proc. Paper 2569).

[8] Superv. Engr., Internatl. Engrg. Co., Inc., San Francisco, Calif.

In view of those discrepancies, and with due cognizance of the many explanations for the same, it should be realized that the analytically obtained stresses may or may not be in reasonable agreement with the true stress conditions in the dam. The theoretically determined maximum stresses represent the general upper limit to which the dam will be actually stressed. With this recognition of limitation of analytical results, these stresses can be used as general indicators that comprise the criteria of acceptance of a design.

Comparison between computed and observed stresses in some of the thinner and recently built USBR arch dams would be of greater interest than results of pioneering studies on the ultra-thick Hoover arch dam to which the author has referred.

Applications of Types of Trial-Load Analyses.—In the writer's opinion, it would be proper to try to establish some limits within which one or the other type of analysis would be adequate to obtain an acceptable design. These limits are suggested as a guide only, and requirements of each structure should be determined on the basis of its individual merits.

Crown-cantilever analysis.—Maximum height of dam up to 150 ft; narrow symmetrical sites; uniformly massive rock foundation, modulus of elasticity exceeding 1,500,000 psi.

Radial deflection analysis.—Maximum height of dam 250 ft; narrow symmetrical sites; massive rock foundation, modulus of elasticity exceeding 2,000,000 psi.

Complete trial-load analysis.—Maximum height of dam exceeding 250 ft; non-uniform, stratified rock foundation requiring substantial treatment, or modulus of elasticity less than 1,500,000 psi; comparatively wide unsymmetrical sites; stage construction effects to be considered. The overall importance of the structure, designer's experience, and other design factors such as temperature changes and joint grouting schedule should also be considered befor the type and extent of analysis are decided.

Analysis Criteria.—Assumptions of temperature change and hydraulic uplift in connection with the design examples presented by the author need some clarification. In each of the cited designs, because of the specified low temperature of concrete at which the contraction joints are to be grouted, the effects of a temperature rise are included in the analysis. Temperature rise counters the tensile arch stresses at abutment extrados and crown intrados, and thus improves the stress condition in the structure. In small arch dams in isolated locations, however, artificial cooling, especially sub-cooling with the aid of an embedded pipe system, may be neither economical nor practicable. In a situation like this the joints will be grouted when the concrete cools to the mean annual atmospheric temperature, and therefore a temperature drop would have to be considered in design. The tensile stresses at arch abutment extrados and crown intrados due to temperature drop may exceed those due to the normal reservoir water pressures. Deflections of the unit arches due to temperature drop can be so high in a slender dam, that virtually all the water pressure will be taken by the cantilever elements. The resultant high vertical tensile stresses at upstream face near the base would crack the cantilever, causing a readjustment in load distribution and the stress pattern. Such an eventuality is often avoided in larger arch dams where rigid temperature control can be justified, as assumed in USBR designs.

It is not clear from the text as well as notes on the figures included in the paper if hydraulic uplift in the concrete or at the foundation is included in USBR

analyses of arch dams. While the effect of uplift on deflection may not be significant, its influence on vertical normal (cantilever) stresses where low compression or tension is indicated at the upstream face, can be substantial.

Allowable Tensile Stresses.—The USBR criteria regarding the magnitude of permissible tensile stresses are unclear. Copen has stated that this would depend on the location of the stress. For obvious reasons, a higher tensile stress can be permitted at the downstream face than at the upstream face of the dam.

The significance of tensile stresses at the upstream face, especially if they are vertical normal stresses or principal stresses almost normal to the foundation surface, deserves careful evaluation. Questions arise if concrete should be assumed to crack when tension exceeds a certain value; or if hydraulic uplift should be treated as an external force before or after the concrete cracks? Complete analysis of Dam A, Fig. 3, shows a maximum tensile principal stress of 222 psi. If this does not include the effect of uplift, would not the total tension be much higher? Supposing the USBR criteria assumes that concrete would crack if tension exceeds 250 psi, would this indicate the likelihood that there will be carcking in Dam A, and consequently the stresses in the dam would undergo readjustment?

Substantial tensile stresses are indicated by complete trial load analyses in several USBR dams.[9] The maximum tensile arch stresses at abutment extrados in some of the dams are:

Dam	Tensile Stress, in psi
Stewart Mountain	285
Horse Mesa	279
Owyhee	242
Mormon Flat	152

It should be interesting to observe the range of actual stresses at the upstream faces of these structures, by installing strain measuring gages on the concrete surface where possible. The purpose being to check the validity of theoretical results, without getting involved in measurement of internal strains and stresses. At many locations in these dams, the computed tension was so high that in the analyses, it was assumed that the concrete would crack. Extensometers installed at the surface, when the reservoir water is low, should be able to indicate if there is any actual cracking or opening of a construction joint when the reservoir fills up. Any measurements or observations along these lines that the USBR may have made would be a further contribution to this subject.

Tangential Shear & Twist Effects.—The difference between a radial deflection analysis and a complete analysis, can be traced mainly to the effects of tangential shears and twisting moments on the deflections and stresses in the dam. These effects vary in significance according to the shape of damsite, height of the dam, and its slenderness or massiveness. In general, inclusion of effects of tangential shears and twisting moments results in:

1. Decreased deflection of the dam in the central portion.
2. Increased compression in vertical normal stresses at upstream face at lower elevations and in central portion. But decreased compressive cantilever stresses at upstream face near abutments.

9 "Treatise on Dams; Chapter 10, Arch Dams," U. S. Bur. of Reclamation, 1956, p. 335-367.

3. Vertical normal stresses at downstream face would reduce in compression in most of the dam, except near the abutments where there may be an increase.

4. Horizontal arch stresses at abutment extrados would reduce in compression in upper portion of the dam but increase in the lower portion. At the crown extrados there would generally be some reduction in arch compression throughout the dam.

5. Horizontal arch stresses at the intrados would show a general decrease at the abutments, but an increase in compression at the crown.

In other words, there is general improvement at most of the critical stress areas, as the author has emphasized.

The magnitude of the tangential shear and twisting moment effects in a dam located in a narrow canyon, such as Dam B, is relatively small. The author confirms this by saying that the radial or complete analyses are likely to indicate little, if any, changes in the design based on a crown-cantilever analysis. In such a situation, especially when benefit of several analyses on other similar dams are available, a complete analysis does not seem justified. After all, the stresses so obtained may still differ from actual condition in the dam as much as they differ from the results of a less detailed analysis. It is with this thought in mind that the writer has suggested the applicability or adequacy limits for different types of analyses previously mentioned.

The tangential shear and twisting moment effects also vary with the slenderness and curvature of the dam. Pronounced curvature in a vertical plane also has an influence on these factors. The author would perhaps like to draw on the vast amount of USBR analysis data and contribute further to this topic.

In his conclusions, the author has mentioned that the trial-load method of analysis is applicable to any type of arch dam. It may be added that with proper modifications, this method has been used for analysis of straight gravity dams where transverse contraction joints are interlocked and/or grouted, and for multiple-arch type buttress dams.

J. A. VELTROP,[10] M. ASCE.—In listing current methods of design in the "Introduction," the author mentions the thin cylinder theory. While the other four methods could be classified as final design methods, the thin cylinder theory hardly fits this category, except perhaps for certain special cases, because it does not even include such an elementary phenomenon as rib shortening. The method of independent arches could have been listed with more justification, although any method of final design should be at least equivalent to the crown cantilever method. The latter deserves separate listing, too, as it is essentially not a trial load method.

Would the author elaborate on his statement in the "Introduction:" "the effective application of simplified analyses have made this method even more effective." Does simplified analyses refer to the crown cantilever method and the radial adjustment method, as illustrated in the paper? The quote seems to imply recent advances. Do these simplified methods now give a better picture of the final stresses obtained by the trial load method than was previously the case?

The author refers to the number of arches and cantilevers used in the trial load method. The necessary number of arches and cantilevers depends pri-

[10] Head, Arch Dam Sec., Harza Engrg. Co., Chicago, Ill.

marily on the accuracy wanted and on the height to width ratio of the canyon. Cantilevers and arches should be placed in such a way as to form nearly square patterns, perhaps, along the vertical cylindrical plane through the axis of the dam.

Much has already been written about the theoretical basis of the trial load method, the uniqueness of its solution, the accuracy of the method, and the agreement of its results with model tests. The statements under the heading, "Basic Principles," about the three adjustments necessary and sufficient to obtain a solution, are not clear. It would seem that three linear and three angular displacements of arches and cantilevers would require six independent adjustments of which two are automatically satisfied by twist adjustment about vertical axis, as presented in a publication[11] cited by the author. It may well be that only the three adjustments, referred to by the author as radial, tangential, and twist, are necessary to obtain a satisfactory solution for the type and shape of arch dams designed by the USBR as compared to model test results. This does not imply, however, that the same procedure would be correct for other shapes, such as thin double curvature arch dams. The author's statement about automatic agreement of vertical displacements by the use of tangential adjustment seems to contradict the requirement of separate vertical adjustment mentioned in another publication.[12] The importance of a separate vertical deflection adjustment for such arch dams has been emphasized by M. Rocha, F. ASCE and J. L. Serafim,[13] F. ASCE. Beaujoint has developed[14] a computer program which includes five independent deflection and rotation adjustments plus a sixth ore for Poisson's ratio.

Are the temperature effects referred to under the heading "Basic Principles" included both in the free deformations of single arches and free standing cantilevers prior to adjustment? From the paper, it appears that temperature deflections are not computed for cantilevers. In the writer's experience, temperature bending deflections of cantilevers are significant for relatively thin double curvature arch dams.

In enumerating the effects included in the elementary theory of flexure for curved beams, no reference is made to the effects of thrust on rotation and of bending on rib shortening. The statement[15] "the arch center line is used instead of the neutral axis" seems to imply that these effects are not included in the arch ring analyses. A paper by O. C. Zienkiewicz,[16] F. ASCE, and the discussion of this paper by the writer,[17] emphasized the importance of this effect on the moments in the arch. Otherwise, crown deflections and arch thrust were not changed.

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11 "Trial Load Method of Analyzing Arch Dams," Bur. of Reclam., U. S. Dept. of the Interior, Boulder Canyon Proj. Final Reports, Part 5, Bulletin 1, Denver, Colo., p. 175.

12 Ibid., 1. 27.

13 Discussion by Merlin D. Copen, et al., of "Model Tests, Analytical Computation and Observation of an Arch Dam," by M. Rocha, J. L. Serafim, et al., Proceedings, ASCE, Vol. 82, No. PO 5, October, 1956.

14 "Le Calcul Global des Barrages-Voutes," by N. Beaujoint, Annales des Ponts et Chaussees, No. 1, January, February, 1960.

15 "Trial Load Method of Analyzing Arch Dams," Boulder Canyon Proj. Final Reports, Bur. of Reclam., U. S. Dept. of the Interior, Part 5, Bulletin 1, Denver, Colo., p. 103.

16 "Comparative Study of a Segmental Arch Ring," by O. C. Zienkiewicz, Proceedings, ASCE, Vol. 86, No. EM 1, January, 1960.

17 Discussion by J. A. Veltrop of "Comparative Study of a Segmental Arch Ring," by O. C. Zienkiewicz, Proceedings, ASCE, Vol. 86, No. EM 4, August, 1960.

The assumption of uniformly distributed radial load (USBR type: unit load No. 1) on the arches in the crown cantilever analysis, as described in the paper, is indeed generally used. It would be equally possible to assume another, also predetermined, distribution and compute the magnitude by the crown cantilever method.

It would be interesting to know whether the USBR has investigated the desirability of introducing such a different radial distribution, for example by studying radial load distributions determined from the radial adjustment and the complete adjustment analyses. How is dead load of the dam handled in the crown cantilever analyses presented in the paper?

The usefulness of the crown cantilever analysis for the preliminary design of arch dams depends greatly on the judgment of the designer. Such judgment can be increased only through experience obtained from performing many arch dam analyses on a variety of types of arch dams and shapes of valleys, and by comparing these results with measurements on the prototype. It would be interesting, in this respect, to have more detailed comments from the author on the allowable stresses for the three arch dams, A, B, and C. What are allowable compressive and, especially, tensile stresses, the latter in arch action as well as at the base of the crown cantilever, at this preliminary stage of design? This would be of value, also, in view of the author's general statements concerning permissible tensile stresses, under the heading "Application of Trial-Load Methods to Design Problems," and the varying values of computed stresses in Tables 1, 4, and 5. How does the author reconcile the need for differences among these criteria at this preliminary stage in view of the different shapes of valleys; wide V, narrow V, and U-shaped. In view of reference to crown cantilever analysis, does the author consider this analysis a minimum for obtaining stress estimates and preparing preliminary concrete volume estimates?

The writer is somewhat surprised by the little time difference between a crown cantilever analysis and a complete radial analysis. It seems to him that many more basic arch and cantilever data are required and that a trial and error adjustment would be much more time consuming. The writer's company has programmed the crown cantilever analysis for water load only. The single operation on the Bendix G-15D takes 30 min to 40 min, depending on the number of arches chosen for the analysis. The radial adjustment analysis, broken up into several steps, takes much longer.

Would the author explain what he means by "as the analysis is completely programmed?" Is it his intension to perform all trial load manipulations on a computer in several steps, with human handling in between, or will the entire trial load analysis be performed in one single operation?

Does the maximum compressive stress limit of 1,000 psi refer to the final design stress? Is the relationship to concrete strength as defined elsewhere.[18]

It would seem that inclusion of earthquake, for example, equal to 0.1 g, in the loading criteria is overly conservative in view of the demonstrated inherent strength of arch dams against overloading. Statically as well as dynamically, it would appear logical and acceptable to the writer to rely on this extra strength during such extraordinary loading conditions, in order to obtain a more economical structure.

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18 "Design Criteria for Concrete Gravity and Arch Dams," USBR, Monograph No. 19, p. 11.

At what stages of the design of a project, the various arch dam analyses, crown cantilever, radial adjustment, and complete adjustment, are conducted, and for what loading conditions at each stage.

It is assumed that the stresses presented in Table 1 are for the loading condition which includes water load, temperature effects, and earthquake. It would be interesting to have the stresses for the case of water load only, and for water load and temperature effects, so that effects of temperature change and earthquake can be studied separately.

While it is true that stresses for the radial adjustment analysis in Table 2 differ from those for the crown cantilever analysis in Table 1, it also appears interesting to the writer that the maximum compressive stresses in the arches at abutment and crown, as well as at cantilever base, are about the same and do not show the decrease the writer would have expected from the refinement of the analysis. The principal stresses in Fig. 3 do, however, show a considerable decrease. Is the lack of decrease of maximum compressive stresses between Tables 1 and 2 normal? The same question is asked for tensile stresses.

The statement that large tensile stresses may occur in the downstream face of cantilever elements has been quoted by many designers, and has been well illustrated by model tests. This is one of the reasons that arch dam designers have gradually adopted the principle of downstream overhang, which precompresses the previous referenced areas.

While agreeing with the author's sentiments on the doubtful accuracy of model results, the writer would like to express similar reservations with respect to analytical solutions, for example, earthquake analysis with a statically equivalent force of a certain percentage of g can hardly be more realistic than Oberti's shaking table. In fact, the writer is inclined to prefer model tests in such a case, over analysis, although he really doubts the need for either in the case of arch dam design.

While the writer agrees with the classification of the trial load method as a superb analytical tool, his satisfaction with the method is tempered by realizing the limitations of the necessary assumptions. It would seem that more measurements on prototype behavior are essential for the further improvement of the economy in the design of arch dams. Creep, non-linear effects, construction control methods, and effects on stresses, all contribute to make the arch designer realize that his job has hardly begun with the analysis of simple loads under idealized conditions, be that on a mathematical model or on a laboratory model.

The improvement of our analytical tools with the advent of electronic computers is impressive and useful. More refined and complicated assumptions can be handled and, perhaps, the computer is setting the stage for the useful application of better fundamental data in the analysis.

ELECTRONIC COMPUTER USE IN SCOPING POWER PROJECTS[a]

Discussion by George Bugliarello

GEORGE BUGLIARELLO,[4] A. M. ASCE.—The authors have indicated that approximately one hundred programs were used in the design of each project on the Columbia River. This information is significant because of the far-reaching implications that it may have on the future development of computer usage in water power engineering.

As the number of programs that are becoming available in this area grows, it will be increasingly possible to design new projects by making almost exclusive use of existing programs. The efficiency in this procedure will depend, of course, on the generality and flexibility built into each program, as well as on the form in which the program is made available, and on the ease with which it can be circulated among users. Further effectiveness can be attained if the programs can be used in series, the output of one becoming the input of the next (for example, the data from an hydrograph program becoming the input of a flood routing study).

Finally, the computing process would be further improved if the programs, rather than being available in a library, could be stored in the secondary memory of the machine and recalled through simple instru[illegible].

From an exploratory analysis, the writer believes that less than three hundred programs would be sufficient for completely automatizing the common operations encountered in hydraulic engineering design, including hydrology. If the same approximate proportion, resulting from the data presented by the authors, between programs in the area of hydraulic engineering and in the other areas is maintained, a total of perhaps a thousand programs should lead to automation of most tasks in the whole area of water resources development design. To be sure, this is a rough estimate, but it may indicate what the computer could achieve in the future in this area.

The development of collections of stored programs, of which the existing program libraries are but the embryonic forerunners, will bring about a sharper differentiation of the degrees of skill that will be required of an engineering staff for communicating with the computer. Basically, three different levels of proficiency should be present in varying proportions among the staff:

(1) Skill in designing the most complex programs, in machine language, so as to maximize the economy and efficiency in the use of the computer. When the programs so designed become of widespread and frequent use, they should be permanently stored and made available to all potential users.

(2) A less exacting degree of skill, sufficient for communicating adequately with the computer, even if not in the most effective way, in the simpler univer-

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[a] January 1961, by P. R. Grimes and G. H. Von Gunten (Proc. Paper 2709).

[4] Asst. Prof. of Civ. Engrg., Carnegie Inst. of Technology, Pittsburgh 13, Pa.

sal languages. This would make possible the speedy construction of programs of contingent value and limited use for which efficiency is an optional matter, that is, a matter of balancing effort and speed in programming versus economy in running. It is important herein to make a distinction between physical and economic limitations (that vary from machine to machine) in solving a problem. Many programs tax a computer to the limit of its capability, thus making skillful programming in machine language mandatory. Others, although feasible in the universal languages, may be excessively expensive to run in these languages. In general, however, programming at this second level of skill should be adequate for a large number of operations. Programs that may become in general demand should, of course, be revised, carefully optimized, and permanently stored and circulated.

(3) Finally, an even lower degree of skill is needed for making use of library and stored programs.

Every engineer will need a good understanding of the computer's potentialities and limitations. An ability to make use of the computer, through training at the third level and, in varying degrees, at the second is important. This will enable him to recall through simple instructions any program that has become part of the permanently stored collection, and also, in a degree commensurate with his proficiency at level two, to construct most additional programs required for his specific needs.

Training at the first level will have to be achieved only by a few specialists on the staff, who should become thoroughly conversant with the programming process and the computer. The specialists should either be hydraulic engineers with a strong background in mathematics or vice versa, because the generation of highly efficient computer programs in the area of water resources engineering (or in any other area) can be achieved only if skill in the use of the computer is coupled with an extensive knowledge of the problems to which this skill is applied.

STRESS ANALYSIS AND SPECIAL PROBLEMS OF PRESTRESSED DAMS[a]

Discussion by Charles Jaeger

CHARLES JAEGER.[4]—A recent discussion by the writer with Zienkiewicz began by talking about tests made for the Allt-Na-Lairige Dam in Scotland and the real significance of these tests as compared to the stresses on the actual dam foundations. The tests were described by J. A. Banks in two papers, one in the Proceedings of the Institute of Civil Engineering, London (10), the other at the Fifth Congress on Large Dams (11). A "Freyssinet" hydraulic jack was anchored in the rock at 18 ft depth underneath the surface, and an isolated force P = 1,000 tons, which subsequently was raised to $P_{max.}$ = 4,000 tons, was applied. At P = 4,000 tons, the Freyssinet jack burst before the rock gave way. A careful examination of the strain-stress curves recorded on the site during the tests showed that rock was plastically deformed during these tests and that shear stresses were probably high.

The writer has tried to analyze these strain-stress curves using Boussinesq's equations. Values of σ_x, σ_y, and τ were obtained. For P = 4,000 tons, they are rather high, and local rupture of the rock near the anchorage was possibly caused by them. On the actual dam site, anchorage is at 26 ft depth and the anchorage force P = 1,176 tons only, cables being anchored at 21 ft spacings.

When the correct numerical values for P, h, b, and d are introduced in Zienkiewicz's and Gerstner's diagrams and equations, the resulting foundation stresses for a dam anchored in rock are low compared with those estimated for the test conditions. During test conditions, tensile stresses in the horizontal direction and shear stresses were particularly dangerous and a multiple of what Zienkiewicz and Gerstner estimated for their case.

By comparing Zienkiewicz's and Gerstner's approach to the estimates made by the writer, the two parts of the complete problem can be compared and the real meaning of the tests by Banks can be correctly appreciated.

When showing that the anchorage computed for P = 1,176 tons at a depth of 21 ft was still holding for P = 4,000 tons at a depth of only 18 ft, Banks had definitely proved his design to be on the safe side (considering that the tests were for an isolated force P). Zienkiewicz's and Gerstner's approach shows the safety factor to be even higher than expected by Banks, so that an anchorage similar to that described by Banks is to a high degree independent of local anisotropy of the rock and local rock fissures.

Obviously the stresses will increase noticeably with higher dams. For higher dams, the anchorage force P (force per unit of dam crest length) will have to be increased, and Zienkiewicz and Gerstner show P/b to be proportional to

[a] January 1961, by O. C. Zienkiewicz and R. W. Gerstner (Proc. Paper 2714).
[4] The English Electric Co., Ltd., Rugby, England.

h, a fact which should be remembered. But by comparison of both the estimates made by the authors and the writer, numerical values can be computed for cases which differ from that presented.

The effort of Zienkiewicz and Gerstner towards solving this particular foundation problem should be considered within the wider framework of "rock mechanics" and "general foundation problems" which are now to be re-examined systematically after some recent dam failures. Another Paper submitted by Zienkiewicz, Veltrop, and Shieh to the Rome Congress is a further effort in a similar direction.

The methods the authors adopt for solving their particular problem should be kept in mind when dealing with other problems concerning dam foundations. The approach used by the authors is general and valid for other cases too.

A Sub-Committee on Underground Works recently created by the International Commission on Large Dams will possibly be the instance where information like that contained in the paper will be centralised for comparison with other publications and so as to be made known to larger audiences.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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c. Discussion of several papers, grouped by divisions.

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